MOMENT CAPACITY RATIO AT BEAM – COLUMN JOINT IN A REGULAR RC FRAMED BUILDING

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CERTIFICATE

This is to certify that the thesis entitled "**Moment capacity ratio at beam – column joint in a regular RC framed building** "submitted by **Sushree Sunayana** in partial fulfilment of the requirement for the award of **Master of Technology** degree in **Civil Engineering** with specialization in **Structural Engineering** to the National Institute of Technology, Rourkela is an authentic record of research work carried out by her under my supervision. The contents of this thesis, have not been submitted in full or in parts, to any other Institute or University for the award of any other degree elsewhere to the best of my knowledge.

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ABSTRACT

Reinforced concrete moment resisting frames (RCMRF) are structural systems that should be designed to ensure proper energy dissipation capacity when subjected to seismic loading. In this design philosophy the capacity design approach that is currently used in practice demands "strong-column / weak-beam" design to have good ductility and a preferable collapse mechanism in the structure. When only the flexural strength of longitudinal beams controls the overall response of a structure, RC beam-column connections display ductile behaviour (with the joint panel region essentially remaining elastic). The failure mode where in the beams form hinges is usually considered to be the most favourable mode for ensuring good global energy-dissipation without much degradation of capacity at the connections. Though many international codes recommend the moment capacity ratio at beam column joint to be more than one, still there are lots of discrepancies among these codes and Indian standard is silent on this aspect.

So in the present work pushover analysis is being done using SAP 2000 for increasing moment capacity ratio at beam column joints and its effect on the global ductility and lateral strength of the structure is studied. To incorporate the uncertainties in material properties, a probabilistic approach is followed to observe the effect of ground motion intensity on probability of exceedance of any specific damage state for structures designed considering different moment capacity ratios (MCR) at the connections. For this objective fragility curves are developed considering the pushover curves obtained from the nonlinear static analysis. Ductility of the structure increases with increase of MCR. Also the buildings designed with lesser MCR values are found to be more fragile compared to the building with higher MCR.

Keywords: pushover, moment capacity ratio, fragility, ductility, lateral strength

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ABBREVIATIONS

RC	Reinforced Concrete
IS	Indian Standard
PGA	Peak ground acceleration
NSA	Non-linear static analysis
FEMA	Federal Emergency Management Agency
MCR	Moment capacity Ratio
ACI	American concrete Institute
Ds	damage state
SCWB	strong column weak beam
ATC	applied technology council
COF	column over strength factor
NZS	New Zealand standard
PA	Pushover analysis

NOTATIONS

DL,LL and EL	Dead load, Live load and Earthquake load respectively
$\mathbf{f_{ck}}$	Characteristics strength of concrete
$\mathbf{f_{y}}$	Yield strength of reinforcement bar
COV	Coefficient of variation
M	Moment of resistance
ds	damage state
$\boldsymbol{\mu}$	Mean Value
$\boldsymbol{\beta}$	Variability Parameter
Gr3 and Gr4	Group3 and Group4 damage state respectively
IO	Immediate Occupancy
LS	Life safety
CP	Collapse Prevention
$\mathbf{V_{B}}$	Design base shear
$\boldsymbol{\Delta}$	Roof displacement

CHAPTER 1

BACKGROUND AND MOTIVATION

1.1 GENERAL

Earthquake is a global phenomenon. Due to frequent occurrence of earthquakes it is no more considered as an act of God rather a scientific happening that needs to be investigated. During earthquake, ground motions occur both horizontally and vertically in random fashions which cause structures to vibrate and induce inertia forces in them. Analysis of damages incurred in moment resisting RC framed structures subjected to past earthquake show that failure may be due to utilization of concrete not having sufficient resistance, soft storey, beam column joint failure for weak reinforcements or improper anchorage, column failure causing storey mechanism. Beam-column connection is considered to be one of the potentially weaker components when a structure is subjected to seismic loading. Figures of some of the beam column joint failure and column collapses in past earthquakes are shown in Fig. 1.1. Therefore such column and joint failure need to be given special attention.

(a)

(b)

(c) (d)

Fig 1.1 Failure of buildings due to storey mechanism in past earthquakes: (a) Failure of column with eccentric connection during Turkey earthquake, 2003 (b) failure of column and beam-column joint during Turkey earthquake, 2003 (c) and (d) Failure building due to column storey mechanism during Bhuj Earthquake, 2001 (Ref www.nicee.org)

1.2 CAPACITY DESIGN CONCEPT

Along with the development of many strength-based design procedures, currently used performance-based seismic design approach of building includes the capacity design philosophy proposed by Paulay and Priestley (1992) as an important tool for earthquake resistant design. In this process the design is based on both the stress resultants obtained from linear structural analysis subjected to code specified design lateral forces and equilibrium compatible stress resultants obtained from pre-determined collapse mechanism. The flexural capacities of members are determined on the basis of overall structural response of a structure to earthquake forces. For this purpose, within a structural system the objects which can be permitted to yield before failure otherwise known as ductile components and the objects which will remain elastic and will collapse immediately without warning known as brittle components are chosen. When the ductile and brittle systems are decided, the design criterion proceeds as follows:

- Ductile components are to be designed with sufficient deformation capacity at least to satisfy displacement-based demand-capacity ratio to have good energy dissipation.
- Brittle components should be designed to achieve sufficient strength levels so as to satisfy strength-based demand-capacity ratio.

So the concept of capacity design attempts to set a strength hierarchy along the load path that aims to ensure that inelasticity is confined in some pre determined and preferred structural components. The load path in a moment frame starts from the slab goes along the beam, beam column joints, columns, foundations and finally the soil below. So the structural elements that are supporting other elements need to be made stronger than the elements being supported by them (only exception is beam-column-joint).The failure modes that results in non-ductile structural behaviour are delayed by providing higher resistances to such modes.

Capacity design procedure first aims at setting the strength hierarchy at member level. So the beam should be designed to have shear capacity more than the limiting equilibrium compatible shear arising at the two ends because of under-reinforced flexural action.

1.3 STRONG COLUMN WEAK BEAM DESIGN (SCWB)

Designing a building to behave elastically during earthquake without any damages will make the project uneconomical. So the earthquake-resistant design philosophy allows damages in some predetermined structural components. One of the most important requirements of the building to withstand any type of earthquakes is not the more force it can resist but the more deformation it can take before complete collapse. Capacity design procedure sets strength hierarchy first at the member level and then at the structure level. So, it needs adjusting of column strength to be more than the beams framing into it at a joint. Mathematically it can be expressed as

$$M_C \geq M_b$$

Where M_c and M_b are moment capacities at the end of column and beam meeting at a joint respectively. The reasons for adopting this SCWB design are discussed below:

- Failure of column will lead to global failure of the structure but if there is flexure failure in beam ends still it can carry gravity loads because its shear capacity is not hampered.
- The beam has to support the floor but column has to take the weight of entire building above it. So failure of the column is more critical than the beam failure.
- Beam with lesser compression loads on them can be designed to be more ductile than columns and absorb large amount of energy through inelastic actions. As the maximum level of displacement loading that may come during an earthquake loading is not known beforehand so the building should be designed such that the ductile that is the under reinforced flexure failure mode precedes the brittle (shear) or non-ductile mode of failure.
- At a particular beam-column-joint if beam is designed to be the weakest then failure of joint (like shear failure of joint core, spalling of concrete, anchorage failure) can also be avoided.

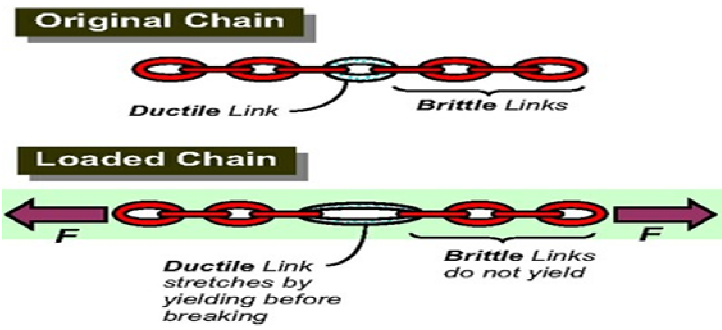
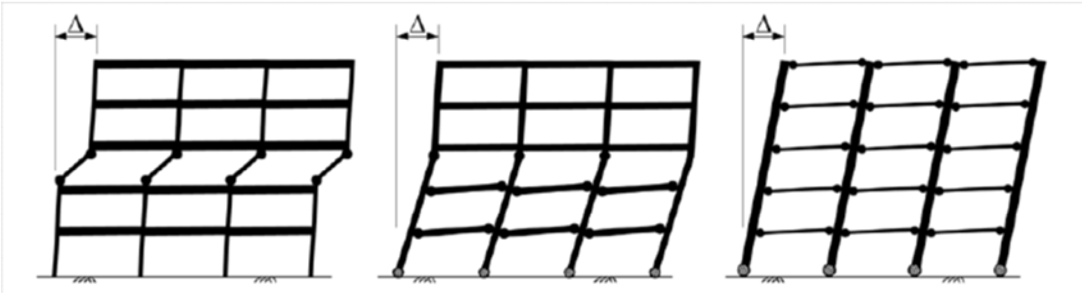


Fig. 1.2 capacity design concept explained by ductile chain analogy (Ref Murty)

- As in ductile chain analogy it is observed that if all the links are brittle and one is ductile and a relative displacement is applied at the ends, internal forces are developed in the links and ultimately the chain breaks when the link with least strength breaks. If this weakest link is ductile type then the chain undergoes large final elongations before fail. So to make a chain ductile the weakest link is made ductile. Similarly to make a structure ductile the weakest component should be the beam as the under reinforced flexure failure and less axial force in beam make it more ductile than columns.
- During an earthquake the inertia force induced in the structure cause it to sway laterally. The distribution of damages over the building height and the lateral drift of the structure are related. Weak columns cause the drift and so the damages to

concentrate in one or a few stories only (Fig. 1.3a), and if the drift capacity of the columns is exceeded then it is of greater consequence. but, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed thus reducing the occurrences of localized damages.(Fig. 1.3c).



(a) Storey Mechanism (b) Intermediate Mechanism (c) Beam Mechanism

Fig 1.3 typical diagram showing distribution of damages and storey mechanism

The utility of adopting this SCWB concept can be illustrated by fig 1.4. The curve shows lateral force versus lateral deformation capacity

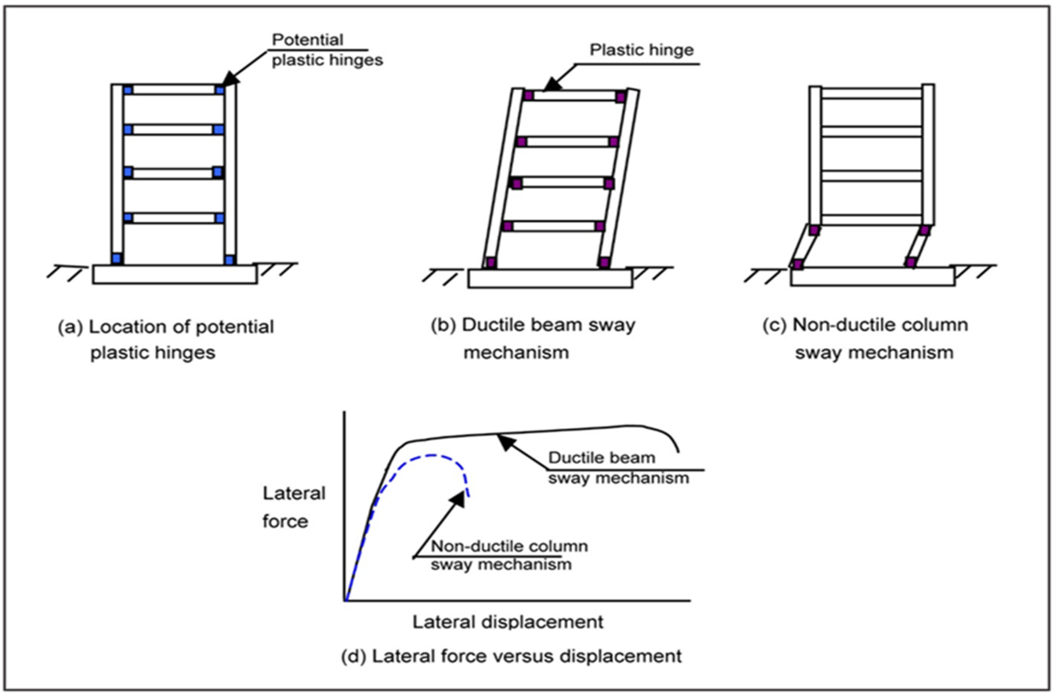


Fig 1.4 comparison of the beam and column failure mechanism by lateral force and displacement curve (refer http://canterbury.royalcommission.govt.nz/Interim-Report-Appendices (29-04-2014)

So from the concepts discussed above it is necessary to adjust the column strength for adequate moment requirements.

1.4 BACKGROUND AND MOTIVATION

In ACI web sessions 1976, when the structure detailed in Fig. 1.4 was being tested for checking the type of joint failure an unexpected result obtained and the beam failed instead of the failure at joint. While investigating this issue the column to beam moment capacity ratio (refer Eq. 1) obtained was more than one.

$$\text{Moment capacity ratio (MCR)} = \frac{\sum M_{nc}}{\sum M_{nb}} \qquad (1)$$

Where M_{nc} = flexural strength of columns framing into joint and M_{nb} = moment capacities of beams framing it.

Hence this concept of moment capacity ratio came into picture. Column–beam flexural strength ratio is certainly an important variable for consideration in overall frame performance. It also determines whether it is the capacity of beam or column that will establish the input force for which joint is designed.

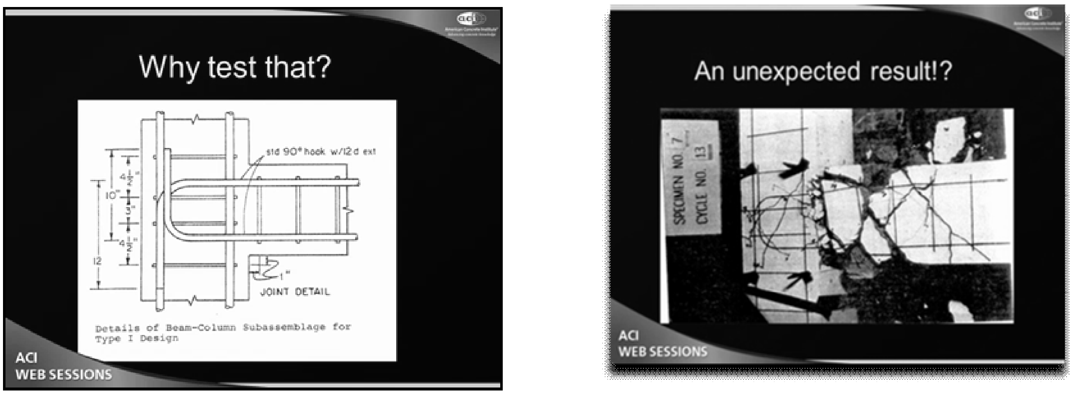


Fig 1.5 Testing details of joint in ACI web sessions 1976

Many international design codes recommend that design flexural capacity of columns framing into the joint is greater than design flexural capacity of beam framing into it. According to some of these codes this ratio varies from 1 to 2. But failure of numerous code-compliant buildings during past earthquake by formation of storey mechanism raises concern on the requirements. There are many discrepancies among these codes. Hence current code provisions are inadequate to prevent column hinges. Also IS codes are silent on this aspect. This is the underlying motivation for the present study.

1.5 OBJECTIVE OF STUDY

Based on the discussions presented above the main objective for the present research is defined as follows:

i. To observe the effect of MCR on ductility and strength of a structure.
ii. The effect of MCR on probability of failure of multi-storeyed building.
iii. To arrive at a moment capacity ratio suitable for Indian Standard.

1.6 SCOPE OF STUDY

The scope of present research work is limited to following structural considerations:

i) Regular RC framed building is selected. Vertical and plan irregularity of the building is kept out of the scope of present study.
ii) The analysis is carried out considering unidirectional lateral loading and thereby only plane frame is considered for the analysis. The results may vary when the lateral load acts along both the horizontal direction simultaneously.
iii) The design of frame sections is assumed to be consistent with the prevailing Indian Standard which ensures no shear failure prior to flexure failure in the frames. Accordingly the nonlinear hinges for shear are not modeled for analysis.
iv) Fixity is assumed in all the column ends.Soil structure interaction is neglected.
v) Only interior joints are considered in the present study

1.7 ORGANISATION OF THESIS

This introductory chapter (Chapter 1) discuss the background, motivation, objectives and the scope of the present study.

Chapter 2 presents literature review on moment capacity ratio of RC and steel building frames, review on various international code provisions, review of literature on pushover analysis and fragility analysis.

Chapter 3 briefly discusses the result and discussions of pushover analysis

Chapter 4 includes the performance assessment of multi-storey buildings considered using fragility analysis

Chapter 5 discusses the summary, conclusions and future scope of the research work.

CHAPTER-2

REVIEW OF LITERATURE

2.1 GENERAL

In the present study literature review is discussed in two parts, out of which first part includes review of various international codes on moment capacity ratio at beam-column joint and the second part deals with the overview on the pushover and fragility analysis of multi-storied RC famed building.

Sugano *et. al*.,(1988) conducted experimental programme on 30-storey RC framed building in Japan and developed design consideration to ensure good collapse mechanism and also observed the ductility of plastic hinges.

Nakashima (2000) observed for steel building that the column over strength factor increases with increase in ground motion amplitude for ensuring column-elastic response. Also for frames in which column-elastic behaviour is ensured, the maximum story drift angle is 1.5 to 2.5 times as large as the maximum overall drift angle.

Hatzigeorgiou(2009) performed an extensive parametric study on inelastic behaviour of reinforced concrete frames under reverse cyclic ground motions and observed the relationship as shown in equation (2.1)

$$\sum M_{n,c} \geq 1.3 \sum M_{n,b} \text{..(2.1)}$$

Jain *et. al.*, (2006) proposed that, when a reinforced concrete moment resisting frame is subjected to seismic loads, at beam-column joint, summation of moment of resistances of columns should be greater than or equal to 1.1 times summation of moment of resistance of beams framing into it as in equation(2.2)

$$\sum M_{n,c} \geq 1.1 \sum M_{n,b} \text{...(2.2)}$$

In this seismic design concept, it is assumed that beams yielding in flexure will precede possible yielding of columns which is recognised as the favourable failure mode (Anderson and Gupta 1972; Clough and Penzine 1982; Park and Paulay 1975; Lee 1996).

According to design provision of Japan (BCJ 2004) a minimum column over-strength factor (COF) of 1.5 is suggested for cold-formed square tube structures in Japan and in seismic provision of structural steel building (ANSI/AISC 341-05) a COF of 1.0 is considered for steel structures. Countries like New Zealand and Mexico adopted a COF ranging from 1.5 to 2.0 (Dooley & Bracci 2001).

Many studies also have been conducted so far by the researchers in search of dominant collapse modes of the frames and designing strong column weak beam frames. Hibino and Ichinose (2005) presented a numerical study of the effect of flexural strength ratio of column to beam on the global energy dissipation of beams and columns in fish-bone-type steel moment frames. The major parameters considered are number of stories, strengths of columns, strengths of beams and ground motion. Findings of the study show that with the increase of the beam to column strength ratio the energy contributing to story mechanism decreases. Nakashima and Sawaizumi (1999) performed dynamic analysis taking ground motion as input in a fishbone shaped model and found that the required COF value that ensures beam hinging responses increases steadily with the increase in ground shaking. Medina and Krawinkler (2005) studied a family of regular frames to evaluate the strength demands suitable for the seismic design of the columns and indicated that the potential of formation of column plastic hinges is high for the frames designed as per the strong column weak beam requirements of current code provisions. Kawano *et. al.* (1998) presented a basic knowledge on the COF for forming the weak-beam type of plastic mechanisms in steel reinforced concrete frames. Dooley and Bracci (2001) investigated the influence of the COF at the joints in two RC frame structures under seismic excitation using inelastic time-history dynamic analyses.

Most of these studies used deterministic approach for specific structures and the probability of undesirable failure mode with risk of failure of the structure remain unknown. Since large uncertainty is associated with the member strength and the earthquake load, the use of probabilistic approach enables the structural safety to be treated in a more rational way. Taking into account these uncertainties, a probabilistic evaluation method (Ono et al. 2000; Zhao et al. 2002) is applied for COF evaluation. For a multi-storey frame the occurrence of potential storey mechanism is very large which increases as the number of stories increases. In another study conducted earlier on storey mechanism of collapse of RC framed structures (Zhao *et. al.* 2007) concluded that all lower storey collapse modes and the upper storey collapse modes with highest failure stories are the frequent occurring failure modes. The least values of COF that ensure probabilistically the entire beam hinging mechanism prior to storey collapse are evaluated.

2.2 REVIEW OF CODES

Some international codes suggest expressions to prevent storey mechanism of collapse due to possible damage locations (hinge formations) in columns. This actually aims at achieving stronger columns with moment capacities more than those of beams framing into a joint obtained considering over strength factors. Moment Calculation at centre of the joint is a very complicated task. These moments are the design moment of resistance of columns or beams calculated at outer faces of the joint and a suitable allowance for moment obtained because of shear developed at the face of joint. However due to such assumptions, the accuracy decreased is very less and the simplification achieved is considerable if the shear allowance is neglected. American and European standard consider this approximation acceptable; Where as New Zealand standard evaluates the moment of resistance of framing members with reference to centre of the joint. In the connections where beams are framing in from both the mutually perpendicular directions, the criteria to find out moment capacities are to be checked independently in either direction.

2.2.1 American Standard

ACI 318M-02suggests that "summation of moment capacities of column sections framing into a joint evaluated at the joint faces considering factored axial loads along the direction of lateral forces resulting in the minimum column moment, should be greater than or at least equal to 1.2 times the moment capacities of the beam sections framing into it.

$$\sum \boldsymbol{M_{n,c}} \geq \boldsymbol{1.2} \sum \boldsymbol{M_{n,b}} \qquad (2.3)$$

In equation (2.3), $M_{n,c}$ and $M_{n,b}$ represent moment capacities of columns and beams framing into a joint, calculated at joint face. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width suggested by the code shall be assumed to contribute to the flexural strength of the beams, if the slab reinforcements are at the critical section for bending. Moment capacities should be considered so that column moments act opposite to the moments acting in the beam end. This equation (2.3) should be satisfied when beam moments act in both directions in the vertical plane of frame considered. If this provision as mentioned in equation (2.3) is not satisfied then the lateral strength and stiffness of columns framing into that joint shall not be considered for calculating strength and stiffness of the structure.

ACI 352R-02 Clause 4.4 recommends For Type 2 connections which are part of the primary lateral load resisting systems, above equation (2.3) is to be satisfied. This verification is not required in connections at the roof level of buildings.

This recommendation that summation of moment capacities of column sections to be greater than the sum of the moment capacities of beam sections (flexural strength under positive bending on one side of the joint plus flexural strength under negative bending on the other side) at a joint is to ensure maximum beam hinging and to reduce likelihood of forming a storey mechanism. The 1.2 factor is to be used when the beam moment capacity under negative bending is obtained considering the effective slab reinforcement participation. This provision does not ensure that the columns will not yield or suffer damage if the structure is loaded into the inelastic range. Studies have shown that higher factors will be needed (on the order of 2 for the uniaxial case and 3 for the biaxial case) to ensure that yielding does not occur in the column particularly if the structure is flexible and higher modes contribute appreciably to the response (Beckingsale 1980; Paulay 1979). The value of 1.2 represents a working compromise between the need to protect against critical column hinging and the need to keep column sizes and reinforcement within an economic range. Tests in which the maximum shear stresses allowed in the joint were used in combination with minimum values of the column-to-beam.

2.2.2New Zealand Standard

The capacity design philosophy requires that for the design of column under flexure moment of resistance of columns should be more than the moment of resistance of beams framing into a joint considering over strength for beams. This requirement is addressed in American standard with reference to the face of the joint. New Zealand Standard (NZS3101:1995) recommends this aspect with respect to centre of the joint as follows:

$$\sum M_{n,c} \geq 1.4 \sum \varphi_0 M_{n,b} \quad \text{...} (2.4)$$

In equation (2.4) φ_0 is over strength factor for beams. The over strength of steel reinforcement is considered as 1.25 and strength reduction factor is taken as 0.85. So the total over strength factor considered for beams is 1.47.The effect of participation of higher modes is taken into account by using dynamic moment magnification factor for enhancing the column moments derived for simply lateral static forces .The value of this factor as suggested by Paulay and Priestley varies from 1.3 to 1.8 in case of two dimensional frames (1992).So the factor 1.4 used in equation (2.4) adopted by the code is thus justified to be in that range. Hence the ratio of moment capacities of columns to moment capacities of adjoining beams at a joint considered is taken as 2.06 (=1.4×1.47). This factor 2.06 is found to be much higher than those recommended by American and European standard..

2.2.3 European Standard

EN1998-1:2003 recommends the following relation between moment capacities of columns to beams that is to be satisfied at all joints:

$$\sum M_{n,c} \geq 1.3 \sum M_{n,b} \quad \text{...} (2.5)$$

In equation (2.5) $M_{n,c}$ is summation of the minimum moment capacities of the columns considering design axial forces and $M_{n,b}$ is summation of the moment capacities of the beams framing into the joint.

2.2.4 Indian Standard

This issue of prevention of anchorage and shear failure in joint region during strong ground motions is not suitably addressed in the design and detailing recommendations for beam-column connections given in Indian standard. In view of these limitations, Jain et al. (2006) proposed a provision in draft for inclusion in IS13920:1993.According to that, In a moment resisting frame, designed for earthquake forces, at a joint summation of the moment capacities of the columns shall be at least equal to 1.1 times the summation of the moment capacities of the beams along each principal plane of the joint.

In another draft for IS 13920(2014), this aspect of moment capacity ratio is considered as follows. At a beam-column connection of a framed structure, the summation of nominal flexural strength of columns meeting that joint (with nominal strength obtained considering the factored axial load in the direction of the seismic force under consideration so as to give least column nominal design strength) along each principal plane shall be minimum 1.4 times the summation of flexural strength of beams meeting at that joint in that same plane. In the case of a beam-column joint not conforming to above criteria, the columns at the joint shall be considered to be gravity columns only and shall not be taken as part of lateral load resisting system. The design moment of resistances of beam shall be calculated on the basis of principles of mechanics and the limiting strain states as per the IS 456:2000.

Mathematically it can be expressed as in equation(2.6)

$$\sum M_{n,c} \geq 1.4 \sum M_{n,b} \qquad (2.6)$$

This check shall be performed at each joint separately for positive and negative direction of shaking in the plane under consideration. The design moment of resistance of columns shall be considered in the direction opposite to the direction of moment of resistance of beams calculated.

2.3 PUSHOVER ANALYSIS

Performance based design philosophy includes the determination of two quantities for design and analysis purpose, one is seismic demand and the other is seismic capacity. Seismic demand is the effect of earthquake forces actually coming on the structure and seismic capacity is the ability of the structure to resist earthquake forces. The performance is evaluated in such a manner that capacity should be more than the demand.

There are so many methods for inelastic structural analysis like linear static analysis, linear dynamic analysis, nonlinear static and nonlinear dynamic analysis procedure. Linear methods are used when the nonlinearity expected is low indicated by demand capacity ratio (DCR) value less than 2.0. Due to some limitations and disadvantages of other methods nonlinear static analysis or push over analysis is considered as the most suitable method for performance based seismic design because it requires less effort and deals with less amount of data for the analysis purpose. Static analysis is proved to be accurate when the structure is short and regular in plan so that higher modes effect are less significant otherwise dynamic analysis is also to be used along with static method.

The use of non-linear static analysis in seismic engineering dates back to the research of Gulkan and Sozen (1974). He used a single degree of freedom system (SDOF) to represent equivalently a multi-degree of freedom structure. The load-displacement curve obtained from this substitution to the real structure was found out by either finite element analysis or manual calculation to obtain the initial as well as post-yield stiffness, the yield strength and the ultimate strength. For analysing multi-degree of freedom systems simplified non-linear inelastic analysis procedures have also been proposed by Saudi and Sozen (1981) and Fajfar and Fischinger (1988).In the context of describing recent advanced developments in earthquake resistant design, Krawinkler (1995) discussed non-linear static pushover analysis as a prelude to capacity spectrum applications. The author mentioned a controversial point, that is, in most cases the normalised displacement profile obtained at a first estimate of the target displacement level is used for defining the shape vectors. The most simple and practical work in this context is done by Sasaki *et. al.*(1996). This work includes running of multiple pushover analyses under forcing vectors corresponding to number of modes that are excited during dynamic response. Out of various modes the mode which will be more effective and cause maximum damage were known if the individual pushover curves converted to capacity curves between spectral displacement versus spectral acceleration using the dynamic characteristics of the individual modes. This procedure is instinctive, and gives advantage over conventional single mode pushover analysis in identifying potential problems.

A brief review done by Tso and Moghadam (1998) showed that fixed load patterns used in pushover analysis have certain disadvantage, but there is not sufficient research to prove the newly proposed variable load patterns as a better option. Kim and D'Amore (1999) compared pushover analysis with inelastic time history analysis. They concluded that all analyses of a structure under a set of specific earthquake motions are not predicted by pushover analysis, a rather obvious conclusion that did not require inelastic dynamic analysis to prove. A single push over analysis under a predefined or fixed loading pattern or displacement vector may not be sufficient to describe the interaction between the continuously-varying dynamic characteristics of an inelastic multi-degree of freedom structure with the various set of natural frequencies.

A modified procedure for pushover analysis was discussed by Bracci *et.al.* (1997). It consists of analysing the structure assuming a fixed lateral pushover load pattern usually triangular. Subsequent increments in loads are calculated from the instantaneous storey resistance and the base shear obtained in the previous step. This method is used in defining the moment curvature relationship of the various members which is used as an input parameter and is utilised to capture the effect of local response. Effect of contribution of higher modes is neglected.

Rana *et. al.*, (2004) performed pushover analysis of a 19-storey building in San Francisco where the plastic rotations of the hinges developed, as calculated by SAP2000, were checked and found to be within the limits suggested by FEMA and ATC guidelines for the intended design objective of Life Safety.

The performance of RC frames was investigated using pushover analysis by Sugani *et. al.*,2012. They observed the performance point *i.e.*, the intersection of seismic demand and capacity curves. Distribution of damages in beams and columns are observed and for properly detailed RC building most of the hinges were formed in beams.

Kihara *et. al.*(2008) observed the column to beam strength ratio of 30 multi storied buildings and concluded that when steel frames are subjected to strong-axis ground shaking, maximum storey drift angle increases for the column-beam strength ratio more than √2.

Poluraju *et. al.*(2011) performed pushover analysis of RC framed structure and concluded that the performance of properly detailed reinforced concrete frame building is adequate as indicated by the performance point and the distribution of Hinges in members show that the desirable collapse mechanism was obtained. Most of the hinges developed in the beams and very few in columns with limited damage. The causes of failure of reinforced concrete building frames during seismic loading may be due to poor quality of the materials .The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures

2.4 FRAGILITY ANALYSIS

Seismic damage assessment can be done by means of fragility curves. Various approaches for damage evaluation, like first-order reliability and a fuzzy random method, were reviewed by F.Colangelo (2008). These methods were used for deterministic analysis of infill reinforced-concrete frame, and the fragility curves obtained were compared. It was concluded that randomness of the capacity is essential. If any damage state is assessed with a deterministic drift range, then fragility sharply increases with increase in peak ground acceleration of ground motion and hence it is quite overestimated.

Seismic fragility curves of RC buildings are studied by Alexander Papailia (2011). The analysis performed for the estimation of the peak response quantities was according to Eurocode 8 (Parts 1 and 3) with certain simplifying assumptions for the frames. The design and the evaluation of the building performance was based on the results of linear elastic (equivalent) static analysis, for a lateral force pattern that is distributed over the height as per an assumed linear mode shape, termed as “lateral force method” in Euro code 8. The median value for the probabilistic distribution corresponding to a given damage state and damage measure of interest was obtained. The dispersion (β value) of the fragility curve was taken into account explicitly the model uncertainty for the estimation of the damage measure given the intensity measure and the uncertainty of the capacity in terms of the damage measure.

Sharfuddin *et. al.* (2010) evaluated the COF requirement that ensure probabilistically the preferable entire beam hinging failure mode and avoid undesirable storey collapse modes of the frame structure during earthquakes. It is found that under same reliability level target COF requirement increases with number of storey and it decreases with the increase in the reliability level.

Milutinovic &Trendafiloski (2003) developed the fragility curves for buildings considered within the RISK-UE project. Building classifications were taken from the stock in the project application sites, *i.e.*, Barcelona, Bitola, Bucharest, Catania, Nice, Sofia and Thessaloniki. Both empirical as well as analytical procedures were employed. Different damage scales were considered for each approach and a correlation between them was proposed. For steel, wood and reinforced masonry structures, the HAZUS vulnerability curves were adopted. Empirical curves obtained using damage probability matrices which were generated from vulnerability index that accounts for structural characteristics and local conditions.

Kappos *et. al.* (2003), within the RISK-UE project, used capacity spectrum method on various configurations of regular RC framed structures designed considering contribution of in-fill and without in-fills (the case of soft ground storey was also examined) for different seismic design levels. Due to assumption of bilinear response in the capacity spectrum method, when there is higher displacement demand than the capacity of regularly in-filled frames with good seismic design, use of the capacity curve for bare frame is recommended. The uncertainty associated in definition of different damage states and the different variability of the capacities were taken from HAZUS. The dispersion for all damage states of a given structural class was the mean of the dispersions for each damage state, so as to obtain non-intersecting fragility curves. Reference was made to the cost of replacement and to a damage index. The vulnerability curves were developed following the hybrid method, where analytical and observational capacity curves are combined.

Akkar *et. al.* (2005) developed vulnerability curves for low-rise and mid-rise infill framed RC buildings. Pushover analyses were performed on 32 existing buildings in Duzce to define the intervals of base shear capacity , period and ultimate drift for two, three, four and five storey buildings with low-level of seismic design. Nonlinear dynamic analyses were performed for 82 recorded accelerograms and bilinear structures with properties within the selected intervals. The effect of number of stories was found significant on the probability of exceeding the moderate and the severe damage limit states. Instead of peak ground acceleration Spectral displacement can be better correlated with peak ground velocity, specifically for higher levels of damage. There was good agreement of the vulnerability curves with observed damage after the 1999 Duzce earthquake.

Erberik (2008) considered 28 RC framed buildings constructed between 1973 and 1999 that were inspected after the Düzce earthquake. Bilinear capacity curves were obtained from non-linear static (pushover) analysis by the distribution of their characteristic properties. 2800 nonlinear dynamic analyses of randomly sampled SDOF structures were also performed for a set of 100 recorded accelerograms. The effect of post-yield to initial stiffness ratio variability (negligible), simulation (negligible) and sampling (negligible) techniques, sample size (negligible), limit state variability (important) and degrading hysteretic behaviour (important) was studied by means of parametric analyses. The analytical curves predicted were in good agreement with the observed damages.

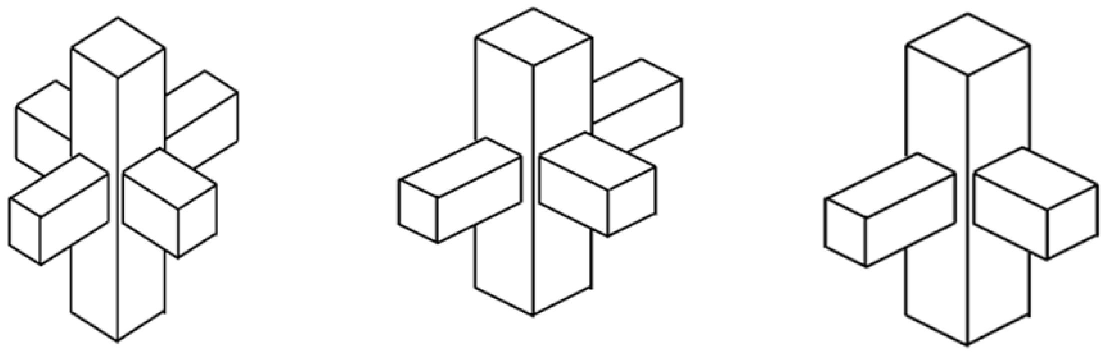
Rossetto and Elnashai (2005) developed fragility curves for low-rise infill RC frames, that were designed and detailed according to Old Italian seismic code. Building design and modelling inputs, uncertainties in the material properties used like randomness in concrete, steel and masonry properties and ground motions data chosen were representative of that region. Adaptive pushover analysis was conducted and a trilinear idealisation of the capacity curve obtained for infill frames were considered. Rather than using graphical method, Nonlinear dynamic analysis was performed for the equivalent SDOF structure and the performance point was obtained. Ten accelerograms records were selected to account for ground motion variability (Wen & Wu, 2001). The results of these analyses were used to construct response surfaces, from which the vulnerability curves were developed considering random values of material properties and spectral displacement to obtain the maximum drift. The analytical curves were in reasonable agreement with empirical curves.

2.5 SUMMARY

In an interior joint two beams and two columns are framing into the joint. So the total beam moment is distributed between two columns. Hence the relation between moment capacities of members framing into an interior joint is as shown in equation (2.6):

$$M(C_1 + C_2) \geq \eta\, M(B_1 + B_2) \qquad (2.6)$$

Where η is the column flexural strength magnification factor.



(a) Interior joint (b) Exterior joint (c) Corner joint

Fig. 2.1: Typical diagram showing beam-column joint

In an exterior joint one beam and two columns are framing into the joint. So, one beam moment is distributed to two columns. Hence the relation between moment capacities of members framing into an exterior joint is as given in equation (2.7):

$$M(C_1 + C_2) \geq \eta\, M(B_1 + 0) \qquad (2.7)$$

In a corner joint one beam and one column are framing into the joint. So the one beam moment is distributed to one column. Hence the relation between moment capacities of members framing into a corner joint is as shown in equation (2.8):

$$M(C_1 + 0) \geq \eta\, M(B_1 + 0) \qquad (2.8)$$

Therefore keeping one constant factor as MCR for all the joints may result in predominant ground storey column failure, which is not preferred.

There are a lot of discrepancies among all the codes regarding the flexural capacity of column to beam at the joint. IS code does not include any such criteria for strong column weak beam design. So it is very important to propose a MCR suitable for Indian Standard

For steel building lots of research are being done in this area in past. But for RC buildings there are not sufficient literature in this area.

CHAPTER 3

PERFORMANCE ASSESMENT USING NON-LINEAR STATIC ANALYSIS

3.1 GENERAL

There are several methods available for analysing a structure. Though elastic analysis gives appreciable result for the determination of elastic capacity of structure and the first yield point but it fails to predict the collapse mechanism and redistribution of forces on subsequent yielding. So to predict the actual response of any structure beyond the elastic limit and identify the potential failure mode when it is subjected to strong earthquake shaking inelastic analysis plays an important role in seismic design. In traditional linear methods lateral force is considered as a basis for design and it does not give accurate result when the ductility demand due to ground motion is high and there is much structural irregularity, On the other hand the nonlinear methods of analysis is based on displacement criteria not the strength. Out of various methods of nonlinear analysis of a structure a simpler option to assess the performance of structures is pushover analysis, even though this also requires as much as possible detailed mathematical models of multi degree of freedom (MDOF) systems. This method is based on the assumption that the response of a structure can be predicted by first, or the first few modes of vibration, which remain constant throughout the elastic and inelastic response of the structure. This provides the basis for transforming a dynamic problem to static one. If the effect of higher modes are significant then modal response spectrum analysis is performed for the structure using sufficient mode to capture 90% mass participation. A computer model of a structure is subjected to a predetermined fixed lateral load pattern, which represents the relative inertia forces experienced by the structure when subjected to earthquake ground vibration. The intensity of the load is increased, i.e. the structure is pushed horizontally and incrementally and the sequence of cracks, yielding, plastic hinge formations, and the load at which failure of the various structural components occurs are recorded. This incremental process continues until a predetermined displacement limit is reached. Thus a force-displacement relationship or capacity curve is obtained, which gives a clear indication of the nonlinear response of the structure.

Pushover analysis (PA) can be considered useful under two situations: When an existing structure has deficiencies in seismic resisting capacity (due to either omission of seismic design when built, or the structure becoming seismically inadequate due to a later up gradation of the seismic codes) is to be retrofitted to meet the present seismic demands, PA can show where the retrofitting is required and how much. In fact this was what PA was originally developed for, and for which it is still widely used

3.2 METHODOLOGY

a) Five, seven and ten storey RC framed (Plane) buildings are designed using commercial software STAAD-Pro.
b) Ultimate flexural capacity of beam ($M_{r,b}$) is determined from the design data obtained.
c) Column reinforcement in the buildings is progressively increased to attain different column to beam moment capacity ratio (MCR) at maximum moment, at zero axial load and at design axial load.
d) Considering the beam and column reinforcement, the same building is modelled using SAP2000 and nonlinear static analysis is being done.

3.3 BUILDING DESIGN AND MODELLING

The present study is based on analysis of a family of reinforced concrete multi-storeyed building frames. These buildings were first designed using STAAD-Pro. The input data required for the design of these buildings are presented in Table 3.1 (a-c).

Table 3.1(a) General building and location details

Type of structure	Multi storey RC frame
Zone	V
Exposure Conditions	mild
Soil type	medium
Damping	5 %
Storey height	3m
Bay width	4m
Design philosophy	Limit State method conforming to IS 456:2000

Table 3.1(b) Details of materials and section property

Beam	300mm× 300mm
Column	300mm×400mm
Concrete	f_{ck}= 25 MPa Poisons ratio = 0.3 Density = 25 kN/mm^3 Modulus of elasticity = 5000 $\sqrt{f_{ck}}$ =25000 MPa
Steel	f_y = 415 MPa Modulus of elasticity = 2×10^{5}MPa

Table 3.1(c) Loadingdetails for the design

Dead load	20 kN/m
Live load	10 kN/m
Equivalent lateral loads	as per IS 1893 (Part I):2002

3.4 PUSHOVER ANALYSIS

From the design of doubly reinforced beam using STAAD, ultimate moment capacity of beam obtained for the five storey building, M_b =136 kNm.

By keeping the beam reinforcement fixed the column reinforcements are increased progressively and buildings are modelled using SAP2000.

Although the hinge properties can be calculated using the reinforcement details by using the concrete models such as confined Mander's model, in the present work the force deformation criteria for hinges developed by ATC 40 and FEMA 273 for concrete and steel have been used in pushover analysis.

Basically a hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads. For example, a flexural hinge represents the moment-rotation relation of a beam of which a typical one is as represented in figure 3.1. ***M*-**

θ_p relation for a section consists of plastic rotation and corresponding moments as ratio of yield moment. This relation affects the behaviour of a section when a hinge is formed there. In the present study all values needed to define M-θ_p relations are obtained by following FEMA guidelines.

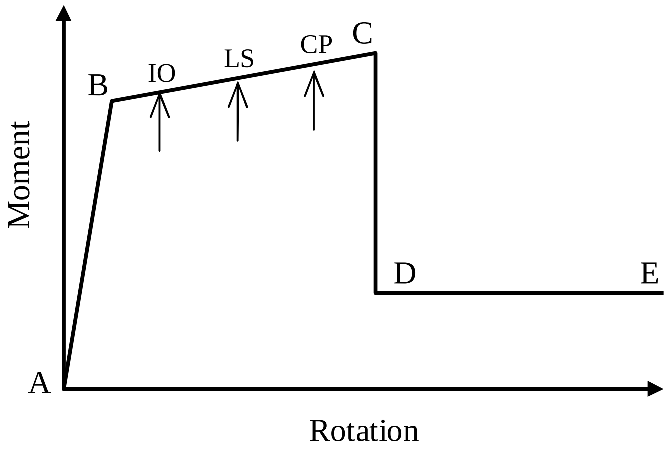


Fig. 3.1 Typical force-deformation curve showing performance levels

As shown in Fig. 3.1, five points labelled as A, B, C, D, and E are used to define the moment rotation behaviour of the hinge and three points labelled as IO, LS and CP indicate the acceptance criteria for the hinge. (IO, LS and CP denote Immediate Occupancy, Life Safety and Collapse Prevention respectively).

Point A denotes unloaded condition.

Point B corresponds to yielding of the element. The portion A to B shows linear response of the structure. The slope of B to C portion is very small and it represents strain hardening phenomenon.

The ordinate at C represents ultimate strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.

The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. D corresponds to the residual strength.

The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, gravity load can no longer be sustained and the strength of the components is reduced to zero. It corresponds to maximum deformation capacity with the residual strength.

The performance of any structure during earthquake depends on the performance of combination of structural and non-structural components. The FEMA 273 defines three structural performance levels and acceptance criteria that relates the earthquake-induced forces and deformations in the structure directly depend on these performance levels which are basically three types as

1. Immediate Occupancy (IO)
2. Life Safety (LS)
3. Collapse Prevention (CP)

3.4.1 Steps used in Pushover Analysis

1. The building is modelled using SAP2000 and the hinge properties are defined and assigned as per FEMA 356 and ATC 40 guidelines.
2. First gravity pushover is applied incrementally under force control for the combination of DL+0.25LL.
3. Then lateral pushover is applied that starts after the end conditions of gravity push over under displacement control to achieve the target ultimate displacement or final collapse.
4. The lateral load pattern to be used in the pushover may be in the form of a specified mode shape, uniform acceleration in specified direction, or a user defined static load case. Here the distribution of lateral force employed is in form of the first mode shape *i.e* the structure is going to vibrate in its fundamental mode.
5. In the model, beams and columns were modelled using frame elements, into which the hinges were inserted. Diaphragm action was assigned to the floor slabs to ensure integral lateral action of beams in each floor.
6. The structure is pushed until global collapse is reached that means when sufficient numbers of plastic hinges are formed to develop a collapse mechanism under the target displacement.

3.4.2 Pushover Analysis Output

The main output of pushover analysis is in the form of base shear versus roof displacement curve called pushover curves. This capacity curve is generally constructed to represent first mode response of the structure assuming that fundamental mode of vibration is predominant. This assumption holds good for structures with fundamental period up to about one second. For more flexible building with fundamental period greater than one then effect of higher modes should be considered. The pushover curves for 5-storey, 7-storey and 10-storey framed buildings are shown in Figs. 3.2-3.4.

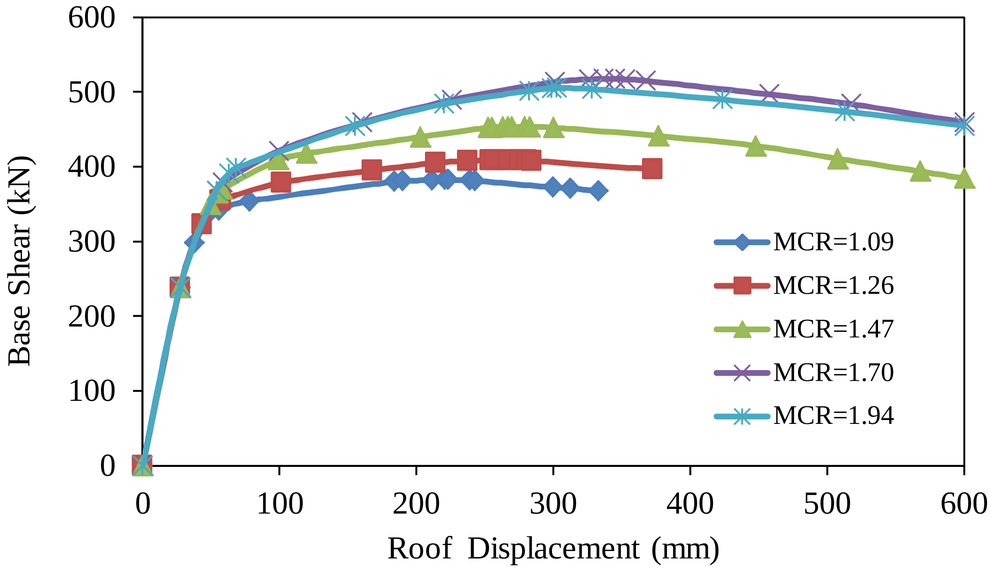


Fig.3.2Pushover curve for 5 storey building frame

These pushover curves show the base shear vs roof displacement for a 5 storey buildings with different MCR values. The MCR for design axial loads are shown in the graph. The initial linear portion indicates the elastic range followed by onset of yielding of steel at one or more plastic hinges. Full yielding of steel (plastic hinge formation) occurs in the inelastic range indicated by the nonlinear portion of the pushover curve. The deformation capacity of the structure for MCR 1.09 and 1.26 are low but then with increase of MCR up to 1.47 shows an increased ultimate displacement that is the target displacement assigned to the structure (4 % of the total building height of 15m). Further increase of MCR does not cause increase of ultimate displacement. After attaining the peak base shear further increase of displacements decreases the strength of the structure and leads to collapse.

Fig 3.3 shows the pushover curve for 7 storey building frame. The curves are initially linear but start deviating as the beams and columns get into the inelastic range. For MCR 1.09 to MCR 1.26 the maximum strength increases but maximum displacement decreases may be because concrete is active in resisting moment. Then with further increase of MCR from 1.47

up to 1.94 the maximum strength does not increase appreciably but the maximum deformation increases.

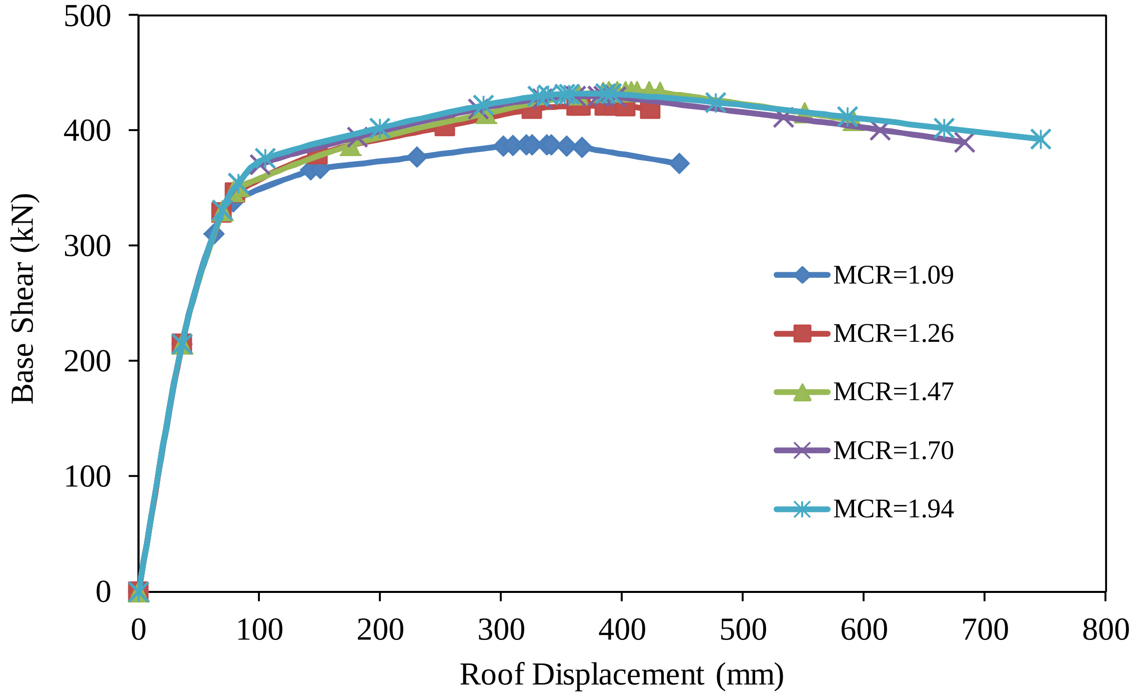


Fig. 3.3Pushover curves for 7-storey building

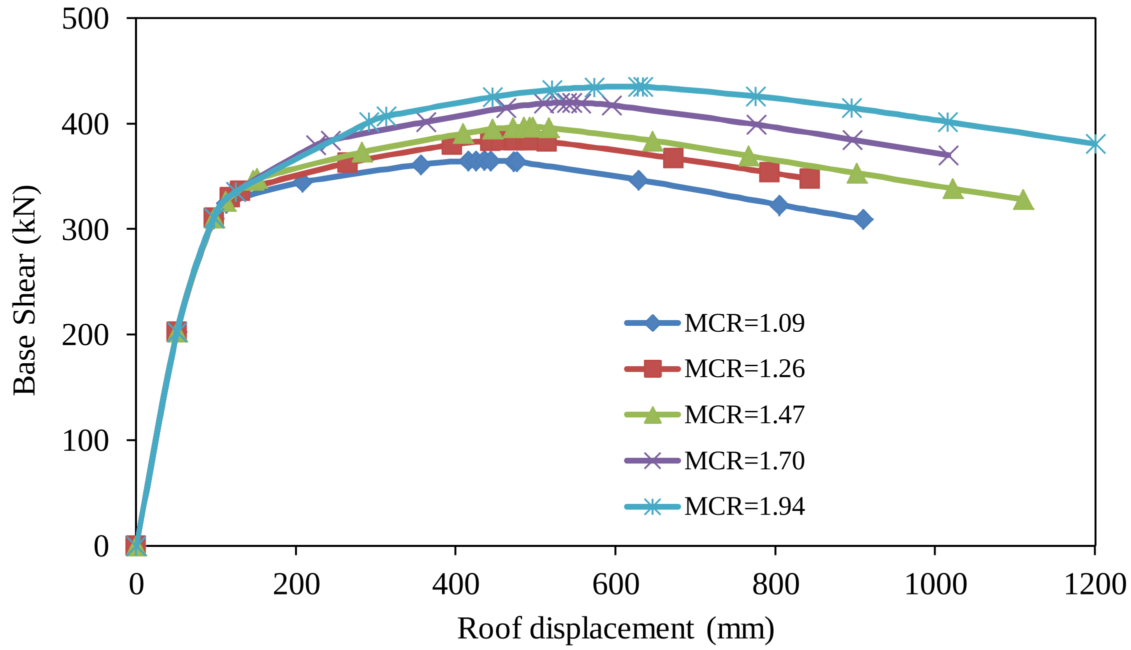


Fig. 3.4Pushover curves for 10 storey building

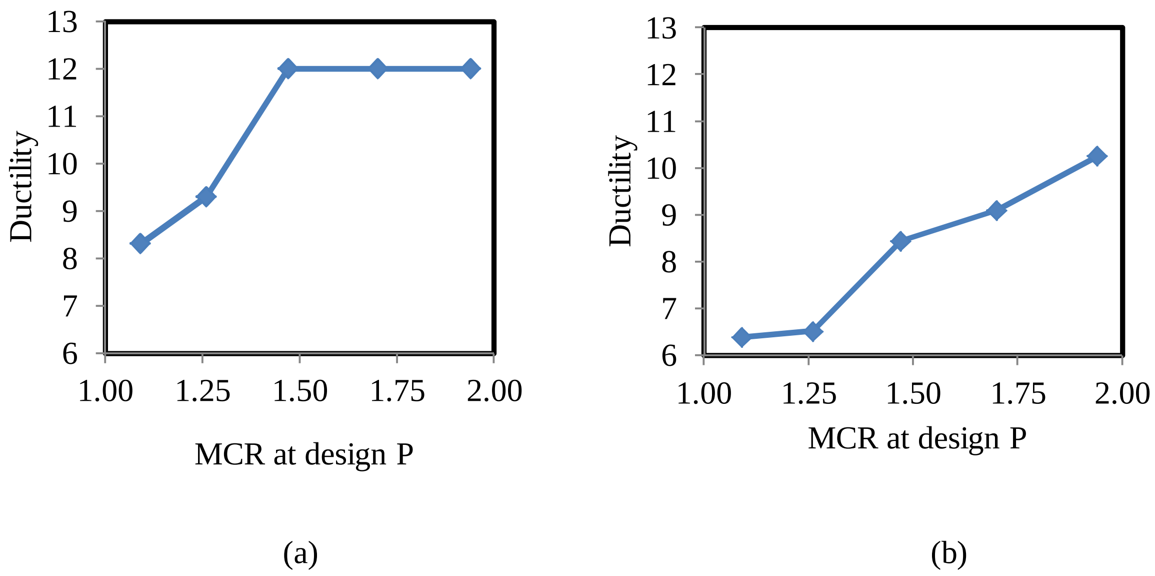
Fig.3.4 shows pushover curves for ten storey framed structure. The curves shows similar pattern under the target displacement. From MCR 1.09 to MCR 1.26 the ultimate deformation decreases slightly and again for MCR 1.47 it increases. For MCR 1.70 the ultimate deformation again decreases and the target displacement is achieved at MCR 1.94 The target displacement is 1.2m (4 % of the 30 m building height).

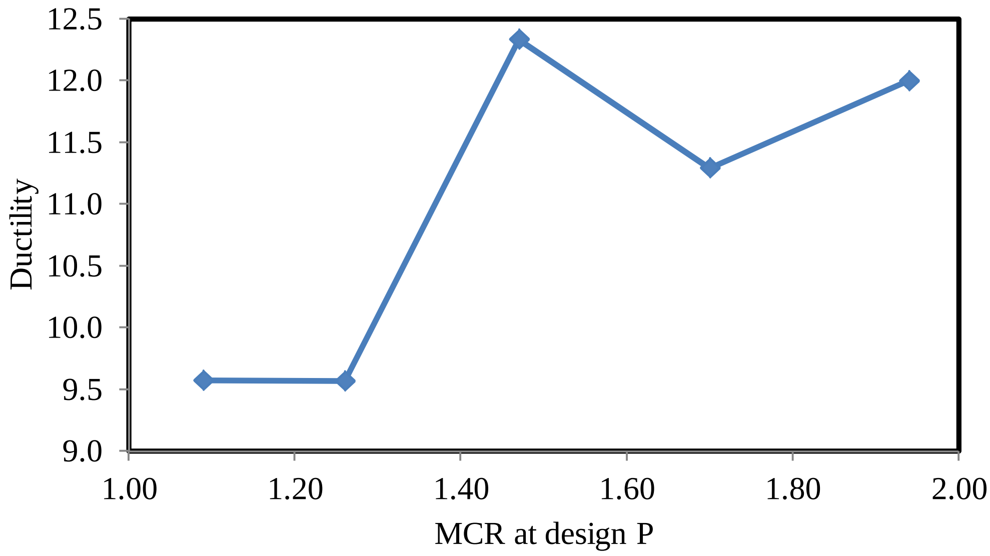
Idealization of pushover curves:

Since the pushover curve is smooth enough hence to get the yield strength and yield deformation and maximum deformation bi linearization of pushover curves is being done. The point where sizable number of components yield that is taken as yield strength and the deformation where the maximum strength degrades to 85 % of peak strength is taken as maximum deformation where the pushover curve shows a dropping portion after attaining peak strength. Otherwise if the curve does not show any descending portion then the maximum displacement obtained from the pushover curve is only considered. Elastic-plastic condition is considered for the idealized curves and equal energy (ensured by equal area under the curve) method is followed for bilinearization.

Ductility as a function of MCR:

From the idealized pushover curve yield point and maximum deformation point can be found out and displacement ductility of the structure is calculated. Displacement ductility is equal to ratio of maximum deformation to yield deformation. Yield strength as well as maximum strength are also found out from the pushover curves.





(c)

Fig. 3 5Ductility as a function of MCR: (a) 5-Storey Building, (b) 7-Storey Building and (c) 10-storey Building.

Fig 3.5(a) shows the MCR versus displacement ductility relation for a five storey framed building. From the curve it is observed that ductility increases with increase of MCR upto 1.47 and for the last two cases it remains the same. This means increase of MCR beyond 1.47 does not contribute to increase in ductility of the structure.

Fig 3.5(b) shows the ductility as a function of MCR for a 7 storey building frame. The ductility increases with increase of MCR from 1.09 to 1.94 but the rate of increase is higher up to MCR 1.47 and after that the rate of increase is comparatively less. Higher the MCR higher the ductility achieved for the seven storey frame considered.

Fig 3.5(c) shows the ductility as function of MCR for a 10 storey building frame. It is observed that ductility increases up to MCR 1.47 and then it decreases suddenly for MCR 1.70 because of decrease in ultimate deformation may be because concrete is active in resisting the moment. Then for MCR 1.94 ductility increases again but the maximum ductility is achieved at MCR 1.47.

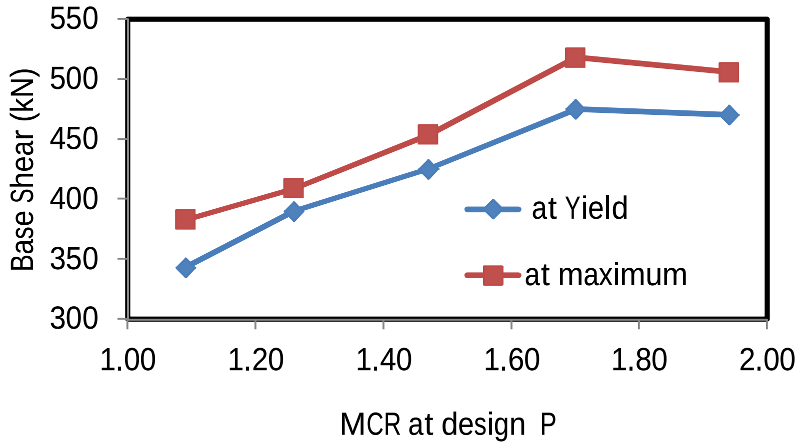
<u>Strength as a function of MCR</u>

Fig 3.6(a) shows the yield strength and maximum strength as a function of MCR for the 5 storey frame. Both the yield strength and maximum strength increases up to MCR 1.70 and further increase of MCR does not increase the strength any more.

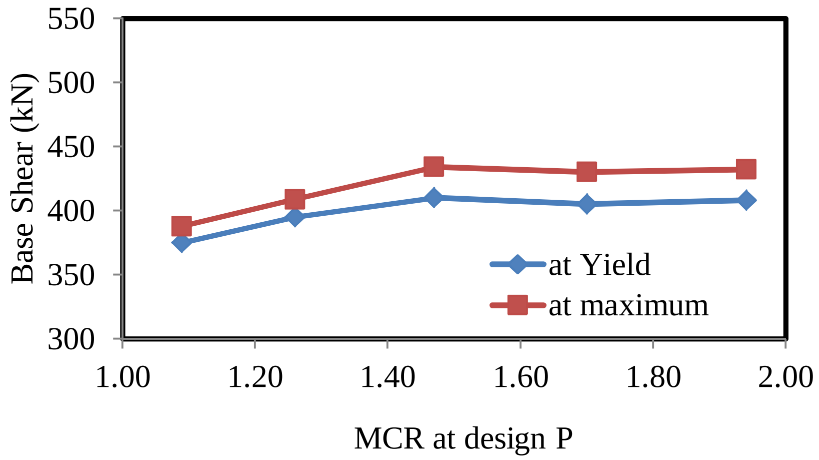
Fig 3.6 (b) shows the strength as function of MCR for the 7 storey frame. It is observed that there is not much effect of increased MCR on the yield strength and also on the maximum strength. However it increases up to MCR 1.47 and after that there is not much effect on strength of the structure.

Fig 3.6(c) shows the strength as function of MCR for a 10 storey building frame. The yield strength shows an increasing trend continuously with increased MCR. The maximum strength increases up to MCR 1.7.

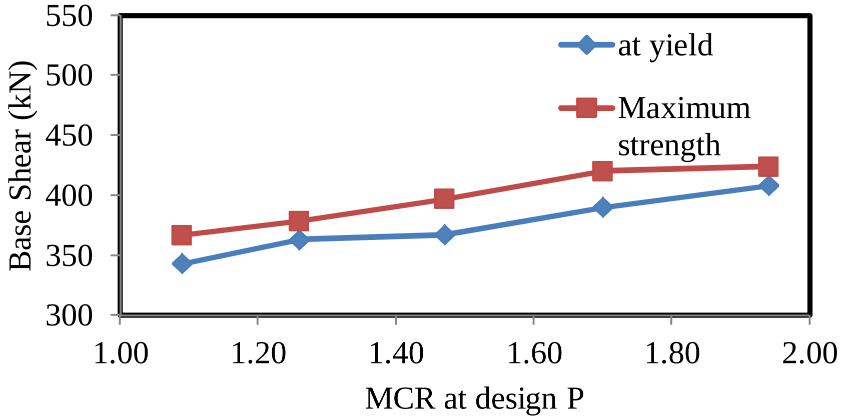
Though higher strength is observed at higher MCR but since earthquake resistant design is not force controlled rather it is deformation based we need to emphasize more on the improved ductility instead of higher strength.



(a)



(b)



(c)

Fig 3.6 Strength as a function of MCR: (a) 5-Storey Building, (b) 7-Storey Building and (c) 10-storey Building

The relation shown in the figures is tabulated in Tables 3.4-3.6.Table 3.4 is for 5 storey frame building.

Table 3.4 Strength and ductility studies for 5 storey building frame

Area of steel(%)	MCR			Yield disp. (D_y) in mm	Ultimate displ. (D_{max}) in mm	Yield strength (f_y) in kN	Max. strength (F_{max}) in kN	Ductility
	At design load	At zero axial load	Max. MCR					
1.01	1.09	0.58	1.17	40	333	343	382.8	8.32
1.18	1.26	0.70	1.29	40	372	390	408.8	9.31
1.57	1.47	0.82	1.53	50	600	425	453.7	12.00
1.96	1.70	1.29	1.64	50	600	475	517.76	12.00
2.36	1.94	1.41	1.76	50	600	470	505.5	12.00

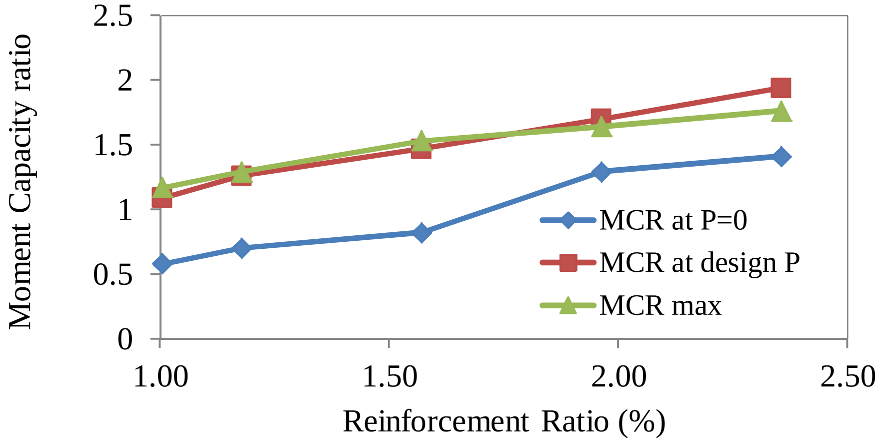
Table 3.5 Strength and ductility studies for 7storey building frame

Area of steel(%)	MCR			Yield disp. (D_y) in mm	Ultimate displ. (D_{max}) in mm	Yield strength (f_y) in kN	Max. strength (F_{max}) in kN	Ductility
	At design load	At zero axial load	Max. MCR					
1.01	1.09	0.58	1.17	70	446.9844	375	387.864	6.39
1.25	1.26	0.7	1.29	65	423.6019	395	408.839	6.52
1.42	1.47	0.82	1.53	70	591.091	410	433.987	8.44
1.94	1.70	1.29	1.64	75	682.875	405	430.135	9.11
2.25	1.94	1.41	1.76	78	800	408	432.153	10.26

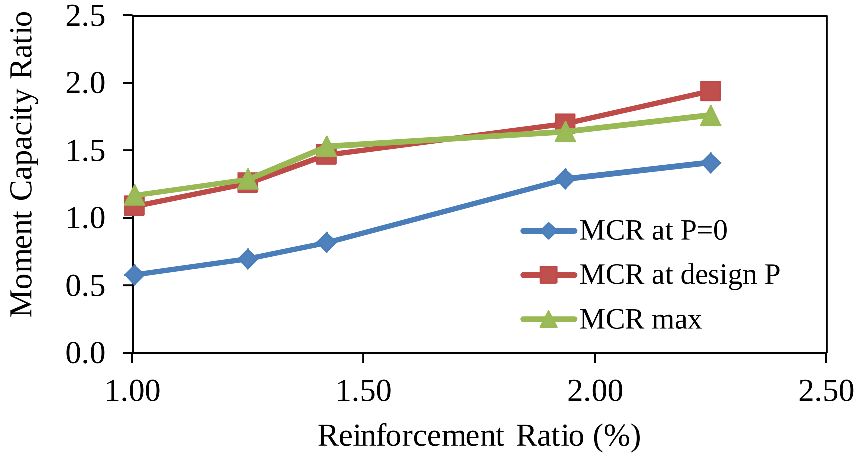
Table 3.6 Strength and ductility studies for 10storey building frame

Area of steel(%)	MCR			Yield disp. (D_y) in mm	Ultimate displ. (D_{max}) in mm	Yield strength (f_y) in kN	Max. strength (F_{max}) in kN	Ductility
	At design load	At zero axial load	Max. MCR					
0.96	1.09	0.58	1.17	95	909.282	343	366.753	9.57
1.25	1.26	0.7	1.29	90	860.867	363	378.635	9.57
1.57	1.47	0.82	1.53	90	1109.991	367	396.976	12.33
1.94	1.70	1.29	1.64	90	1016.213	390	420.148	11.29
2.50	1.94	1.41	1.76	100	1200	408	423.845	12.00

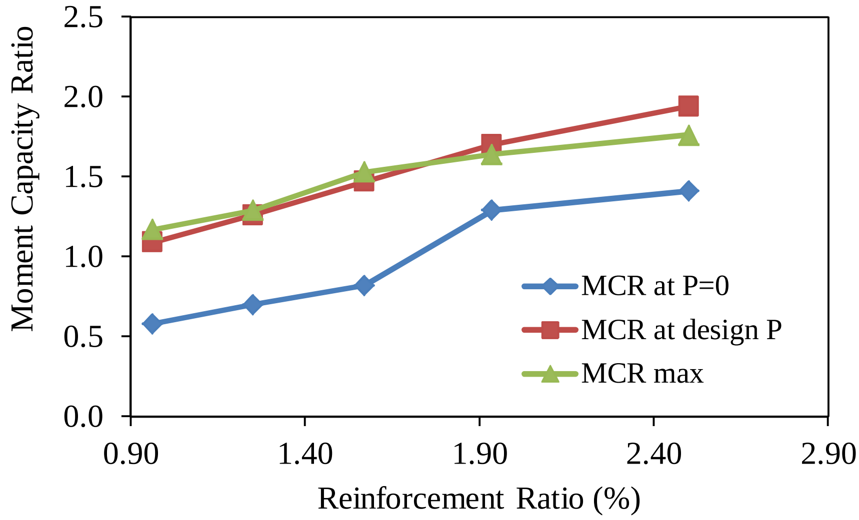
Fig. 3.7 presents the reinforcement ratio (ratio of the area of longitudinal steel to the gross cross-sectional area of the column section) as a function of MCR. The minimum and maximum permissible longitudinal reinforcement in column is 0.8% and 6.0% of the gross cross sectional area respectively. Fig. 3.7 (a-c) shows MCR as a function of reinforcement ratio.To arrive at a suitable MCR for achieving overall ductility of the structure the reinforcement provided should be well within the allowable limit. Also, these figures can give an approximate relation between the economy and MCR.



(a)5-storey building frame



(b) 7-storey building frame



(c) 10-storey building frame

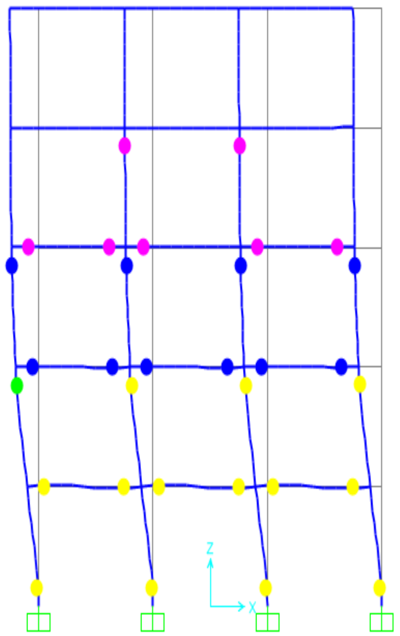
Fig 3.7 MCR as a function of reinforcement ratio

3.5 FAILURE MECHANISM

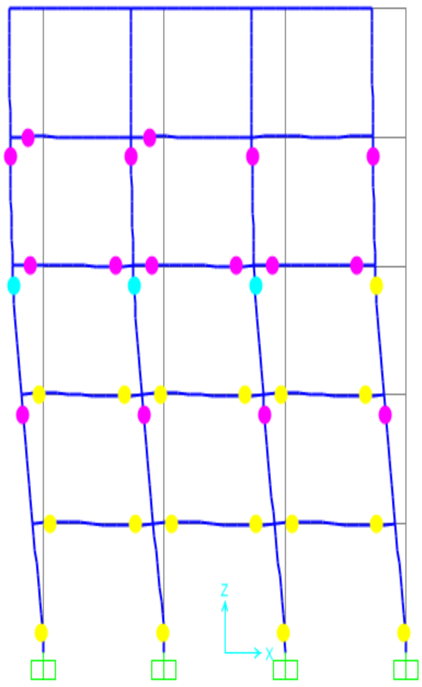
Under the application of pushover loads the members in a structure initially remain elastic up to a certain moment M_p that is the maximum moment of resistance of a fully yielded section. Any further increase of moment will cause the beam to rotate with little increase in load. The rotation occurs at that particular moment M_p. So these expected locations of damage caused by a yielded zone having large inelastic rotation capacity at constant restraining moment M_p are called plastic hinges. The combination of inelastic hinges at the ends of beams and columns which when formed in a building eventually makes it unstable and causes its collapse, hence called collapse mechanism. A desirable collapse mechanism can enhance the ductility of a structure. The hysteretic loop implies good energy dissipation in the building through each of the inelastic hinges at the beam ends. Such behaviour is observed in a building which fails in sway mechanism that ensures the beam yields before column and columns are made stronger than beam. The mode of failure in the form of sequence, location and number of plastic hinges in a 5 storey building is as shown below in figure 3.8(a-e). As we go on increasing MCR the hinge distribution are of more preferred type that is less inelastic actions in columns.

The plastic hinges should be distributed throughout the structure for a good failure mode so that maximum number of members will be involved in energy dissipation. If the damages are concentrated only at few locations then members in those locations will collapse even before the other members get into the inelastic range. So the structure will not get advantage of presence of those members in energy dissipation. So sequence and distribution of hinge formation is very important to have a good failure mode in the structure.

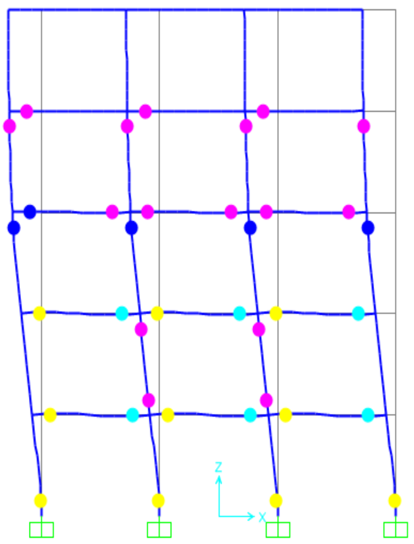
Plastic hinge formation showing different failure mechanisms are obtained considering different MCR values. The hinge formations started from the lower storey and proceeds to upper stories when more components take part in the energy dissipation with progressive increase of MCR. The final step of hinging at failure after attaining the target displacements are shown in the figure below.



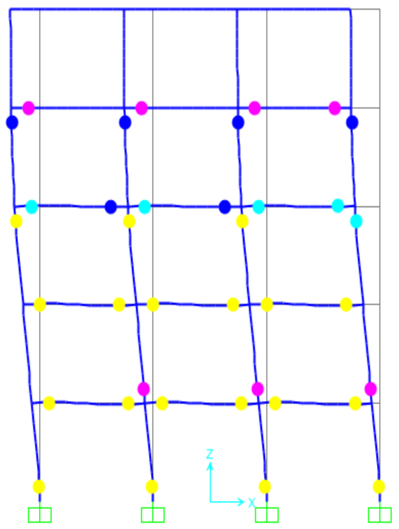
(a) MCR= 1.09



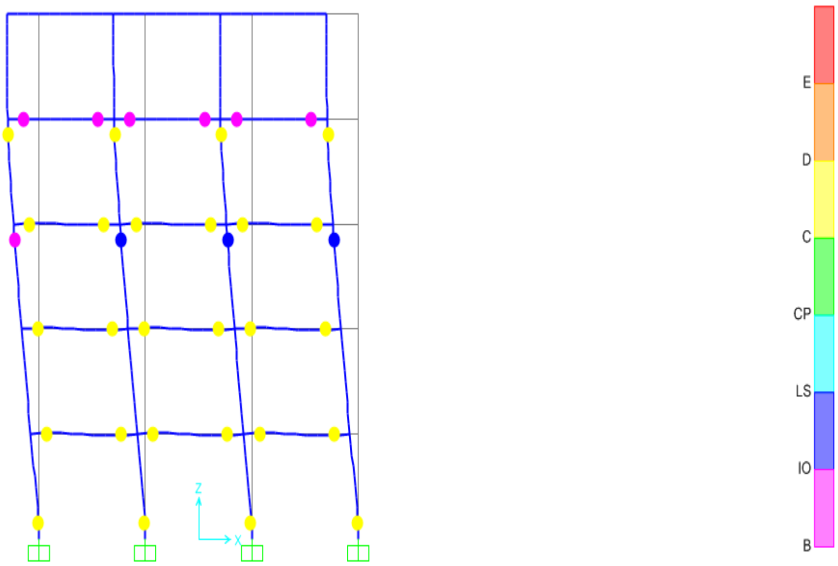
(b) MCR = 1.26



(c) MCR=1.47



(d) MCR=1.7



(e) MCR=1.94

Fig. 3.8 Distribution of hinge formation at collapse for different MCR for the 5storey building considered.

3.6 SUMMARY

1. Plane building frames are designed with IS-456:2000 for loading requirement of IS-1893:2002 and IS-875 (Part-1, 2) using STAAD-Pro for varying MCR.
2. Nonlinear static analysis is being carried out to understand the effect of MCR in the response of framed building.
3. It is found that with increase of MCR at design axial load upto 1.47 for uniaxial bending in a plane frame improves the ductility at an expense of extra reinforcement, with further increase of MCR there is not much increase in ductility. Increase in strength either at yield or maximum is not very significant with progressive increase in MCR for a seven storey building frame. but for 5 storey and 10 storey frames strength also increase significantly upto MCR 1.7.Since seismic design philosophy aims to achieve good ductility in a structure so we need not have to think for higher strength but for higher ductility.
4. A preferable collapse mechanism can be achieved by increasing MCR.

5. Providing a high MCR value need not necessarily imply that formation of column hinges is completely avoided because to completely prevent storey mechanism MCR may be too high but may not be that economical.
6. The suitable MCR must represent a compromise between the need to protect against critical column hinging and the need to keep column sizes and reinforcement within an economic range.

CHAPTER 4

PERFORMANCE ASSESSMENT USING FRAGILITY CURVE

4.1 GENERAL

Most of the studies regarding performance based seismic design are based on deterministic approach. But since lots of uncertainties are associated with material strength and earthquake loads so a probabilistic approach seems to be a more rational way for performance assessment of a structure. This chapter focuses on the background information regarding formulation of fragility curve and the methods used for its development.

Seismic vulnerability or fragility of a structure is defined as its susceptibility to damage by the earthquake loading of a given intensity. The fragility curves can be regarded as one of the most useful tool for performance based design of structures for design of new buildings and also for assessing performance of existing buildings situated in the earthquake prone area all over the world. Evaluation of damage state probability is very important in estimating earthquake losses. It is expressed as probability of attaining or exceeding a certain damage state in terms of ground motion severity that may be PGA or spectral displacement. A number of approaches are available for developing the fragility curves for different types of building considering either the empirical data from past earthquakes or using the data obtained from analytical simulations. In the present work HAZUS methodology is used to develop fragility curves for the 5-storey,7-storey and 10-storey building frames considered. This method considers spectral displacement or acceleration as the ground motion parameter unlike the methods recommended by ATC which describes ground motion in terms of PGA.HAZUS damage functions for ground shaking includes 2 components:1) capacity curve based on engineering parameters like yield strength and ultimate strength obtained from pushover analysis 2) fragility curves describing probability of damage to building for four physical damage states such as slight, moderate, extensive and complete damage states.

For ductile frame structures considered the assumptions made are as follows:

- Elastic-plastic frames are considered.
- All the random variables are statistically independent of each other and are assumed to follow log normal distribution pattern.

- Geometrical second-order and shear effects are neglected. The effect of axial forces on the reduction of moment capacities is also neglected.
- All beam-column connections have identical MCR, i.e., there is only one value of MCR for a structure.

4.2 DEVELOPMENT OF FRAGILITY CURVES

Fragility curves may be defined as the log normal distributions representing the probability of attaining or exceeding a given structural or non structural damage state with median estimate of spectral response (spectral displacement in the present work) being known. This is mathematically expressed as shown in equation (4.1)

$$P[d_s/S_d] = \emptyset[\frac{1}{\beta_{ds}} ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)] \text{..(4.1)}$$

Where φ is known as normal cumulative distribution function.

β_{ds} is the variability parameter obtained from standard deviation of natural logarithm of the spectral displacement for damage state ds.

$S_{d,ds}$ is the median spectral displacement at which building reaches the threshold of damage state ds.

4.2.1 Variability parameter(β_{ds})

$$\beta_{ds} = \left\{\left(CONV\left[\beta_C, \beta_D S_{\bar{d,ds}},\right]\right)^2 + \left(\beta_{M(ds)}\right)^2\right\}^{1/2} \text{..................................(4.2)}$$

In the equation (4.2) β_c= lognormal standard deviation parameter showing variability in capacity properties of the building

β_D= variability in the demand spectrum due to spatial variability of the ground motion.

$\beta_{M(ds)}$ = uncertainty in the estimation of the damage state threshold.

The total variability of structural damage can be taken as combining the three damage variability given in the above equation using the complex convolution process. However HAZUS has defined uniform moderate variability for damage state threshold ($\beta_{M(ds)}$) as 0.4 and capacity curve variability (β_c) as 0.3. The variability due to post-yield degradation for Gr3 damage states considering minor degradation is 0.9 and for Gr4 damage states, considering major degradation is 0.5. So the total variability (β_{ds}) for Gr3 damage state is taken as 0.75 and Gr4 damage state is taken as 0.85.

4.2.2 Damage states:

Damage states give an idea of building physical conditions which is related to various loss parameters like economic loss, functional loss etc. The damage states defined by Barbat *et al.* (2006) based on yield and ultimate spectral displacements of a building are used in the present work. This is shown in the table 4.1 given below.

Table 4.1 Damage State definition (Barbat *et. al.*,2006)

Damage grade	Damage state	Spectral displacement
Gr1	Slight damage	$0.7S_{dy}$
Gr2	Moderate damage	S_{dy}
Gr3	Extensive damage	$S_{dy}+0.25(S_{du}-S_{dy})$
Gr4	Complete damage	S_{du}

4.2.3 Building Capacity curve

Capacity curve can be represented as a plot of lateral resistance of a structure as a function of lateral displacement. It is derived from the pushover curve obtained from nonlinear static analysis in the form of base shear versus roof displacement . Capacity curve for every model building can be obtained for different level of design for a given loading condition and specific performance level.

The yield and ultimate spectral displacements are obtained from the bi-linearization of capacity curves. Yield capacity indicates the lateral strength of a structure. The yield spectral displacement can be taken as the displacement where significant loss of stiffness of the structure occurred due to yielding of maximum number of members. Up to the yield point the curve is linear with stiffness depending on expected time period of a structure. From yield capacity to ultimate capacity there is change of slope indicating the development of plastic stage. After the ultimate point the building's lateral load resisting capacity decreases appreciably. The ultimate spectral displacement is taken as the maximum displacement where strength of a structure decrease to 85 % of the peak strength as considered in this present study.

Figure 4.1 shows a typical building capacity curve as per HAZUS.

Where C_s is point of significant yielding of design strength coefficient; T_e is the expected elastic fundamental mode period of the building; α_1 is fraction of building weight effective in pushover mode; γ is the over strength factor relating true yield strength to design strength ;λ is over strength factor relating ultimate strength to yield strength and μ is the ductility ratio relating ultimate displacement to λ times yield displacement

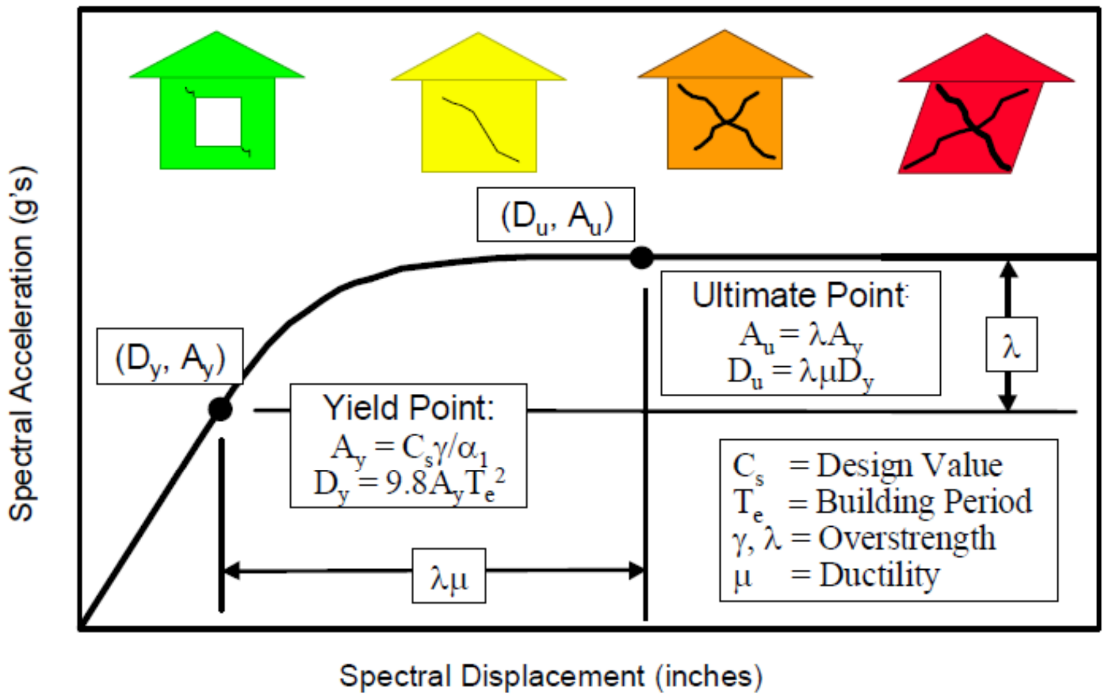


Fig. 4.1A typical building capacity curve with performance (HAZUS, 2003)

4.2.4 Sampling

There are many uncertainties associated with the material properties of concrete and steel used in any construction In the recent years, due to absence of actually observed experimental data many analytical methods have been adopted for generating the numerous data those are required for development of fragility curves. These analytical techniques save time, reduces cost and controls number of data. So to generate number of data to incorporate these uncertainties, Latin hypercube sampling method is used in the present study. This method has the advantage that it requires lesser number of simulations and has smaller sample size in the analysis process. The capacity of any members or the system as a whole depends upon material strength which is inherently uncertain. This uncertainty can be modelled by using two statistical parameters like mean and standard deviation to define the central value and variability.

Table 4.2 shows mean and covariance of the random variables considered. The values presented for concrete and steel are referred from the paper by Ranganathan(1999).MATLAB program is used to do the sampling.

Table 4.2 Details of random variables used in LHS scheme

Material	Variable	Mean(μ)	COV(%)	Distribution
Concrete	f_{ck}(MPa)	30.28	21	Normal
Steel	f_y (MPa)	468.9	10	Normal

4.3 BUILDING MODELLING AND ANALYSIS

20 buildings are considered for fragility analysis corresponding to each MCR value. Non-linear static analysis(pushover) is carried out using SAP2000. This pushover analysis method is mostly used to obtain quantitative limit state values. The critical points like yield and ultimate response and initiation of a collapse mechanism are obtained from the pushover curves (in the form of base shear versus roof displacement) using bi-linear idealization.

Table 4.3 Median Spectral displacement (mm) corresponding to different damage grades

MCR	1.09		1.26		1.47		1.70		1.94	
Damage States	Gr3	Gr4	Gr3	Gr4	Gr3	Gr4	Gr3	Gr4	Gr3	Gr4
5- storey	97	253	184	600	186	600	195	600	198	600
7-storey	160	430	170	441	215	618	260	800	263	800
10-storey	286	847	340	1059	354	1085	358	1074	397	1200

Using the Table 4.3 median spectral displacements for different damage states are obtained. Only damage states of Gr3 and Gr4 are considered in the present study for developing fragility curves. From the spectral displacements obtained for 20 cases median spectral displacement ($\bar{S}_{ds}$) are obtained. Median spectral response shows the threshold limit of a given damage state. Then using the normal distribution function probability of equal or exceeding a given damage state can be obtained.

4.4 PERFORMANCE OF 5-STOREY 3-BAY BUILDING FRAMES

Fragility curves for 5-storey 3-bay framed building is developed as per methods discussed above for different MCR values for the two damage states Gr3 and Gr4.

The slope of fragility curve developed depends on the log normal standard deviation value β. Smaller value of β indicates lesser variability of damage state and hence steeper fragility curve is generated. So the Gr3 curves are stiffer than Gr4 curves (β of Gr3 = 0.75 and for Gr 4 it is 0.85).

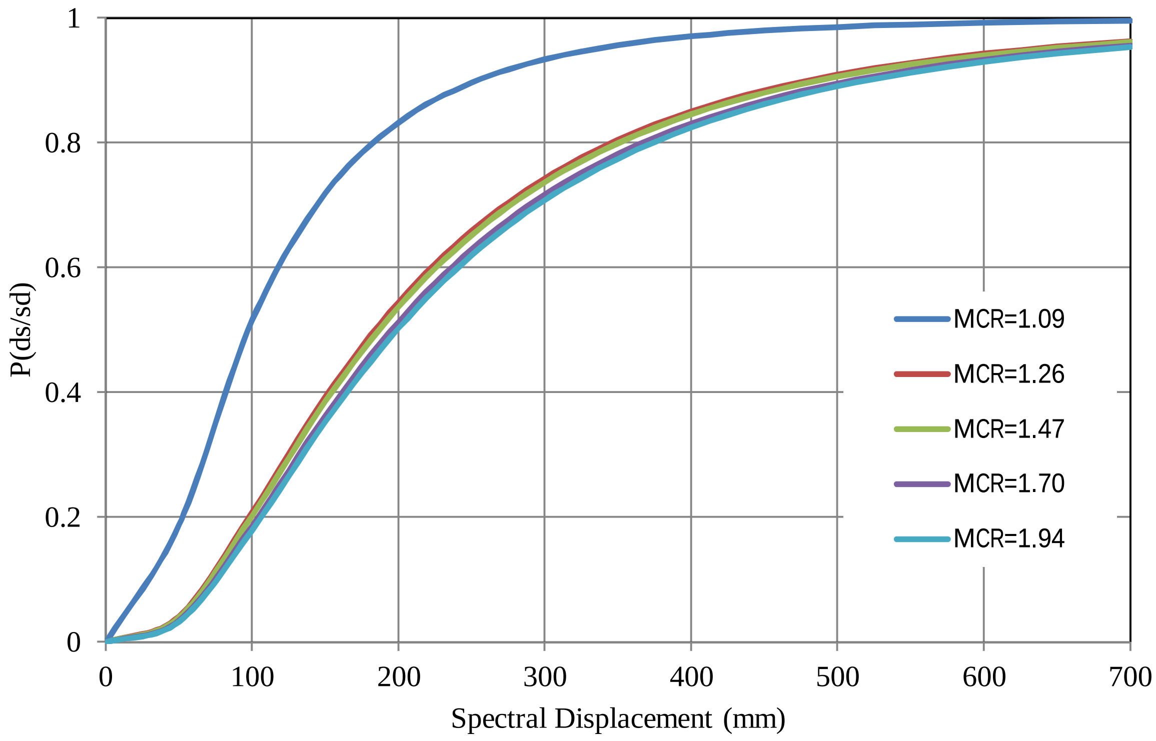


Fig. 4.2(a) Fragility curves for 5-storey framed building for Gr3 damage state

Fig. 4.2(a) shows fragility curves for 5-storey frames at different MCR values for Gr3 damage state. It is observed that the building designed for lesser MCR is more fragile than the same building designed with higher MCR. So for MCR 1.09 the structure is most fragile. For a assumed spectral displacement of 200 mm with increase of MCR to 1.26 reduces the probability of exceedance from 83 % to 55 %. There is a wider difference of fragility with increase of MCR from 1.09 to 1.26.With further increase from MCR 1.26 to 1.94 there is not much decrease in the fragility of considered building.

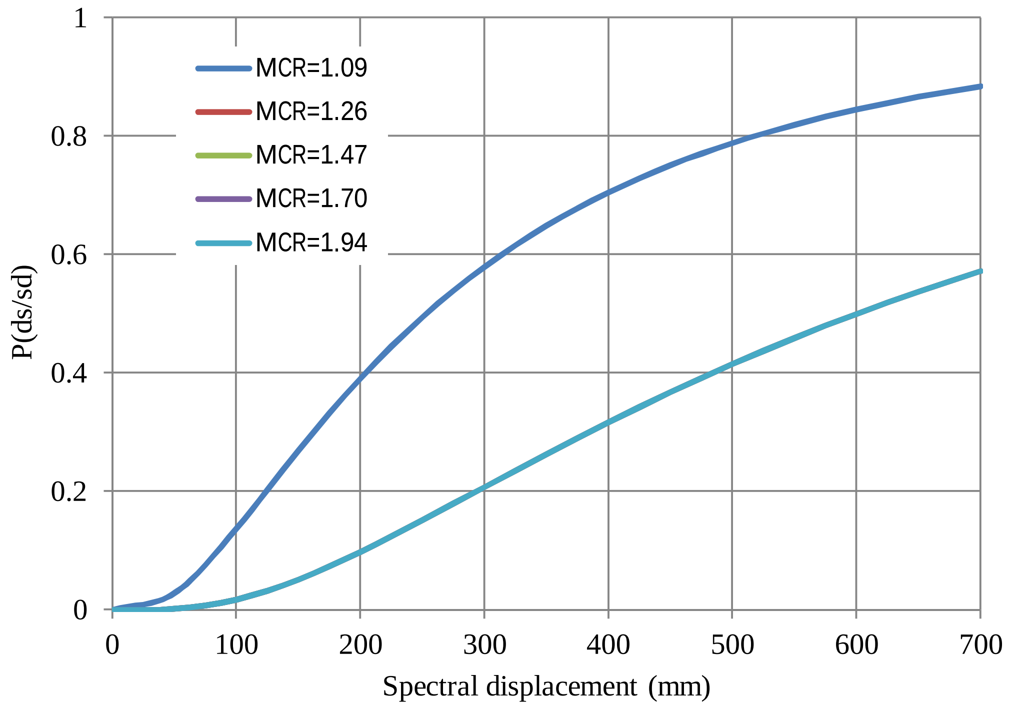


Fig 4.2(b) Fragility curves for 5-storey framed building for Gr4 damage state

Fig. 4.2(b) shows the fragility curve for 5-storey building frame for Gr4 or complete damage state in terms of probability of exceedance and spectral displacement. This curve also shows the similar pattern as Gr3 damage state curve. The building with MCR 1.09 is most fragile and with increase of MCR to 1.26 the probability of exceedance of the specified damage state decreases to a greater extent. However, beyond MCR 1.26 there is no change in the fragility curve because of same median spectral displacement. Since for a given damage state all other parameters being constant the probability of reaching or exceeding that state depends only on the median spectral displacement.

4.5 PERFORMANCE OF 7-STOREY 3-BAY BUILDING FRAMES

The trend in the fragility curves with increase in number of storey in a building may be different. So to consider this, fragility curves for a seven storey building frames with increasing MCR values are developed. Fig 4.3(a) shows the relationship between probability of exceedance and spectral displacement for 7-storey building for Gr3 damage state. It is observed that the building designed with higher MCR is less fragile. Suppose for a S_d of

200mm the probability of exceedance is 62 % for MCR 1.09,60 % for MCR 1.26,45 % for MCR 1.47,38 % for MCR 1.70 and almost same for MCR 1.94. There is greater difference in fragility from MCR 1.26 to 1.47 but then to 1.70 the decrease in fragility is comparatively less.

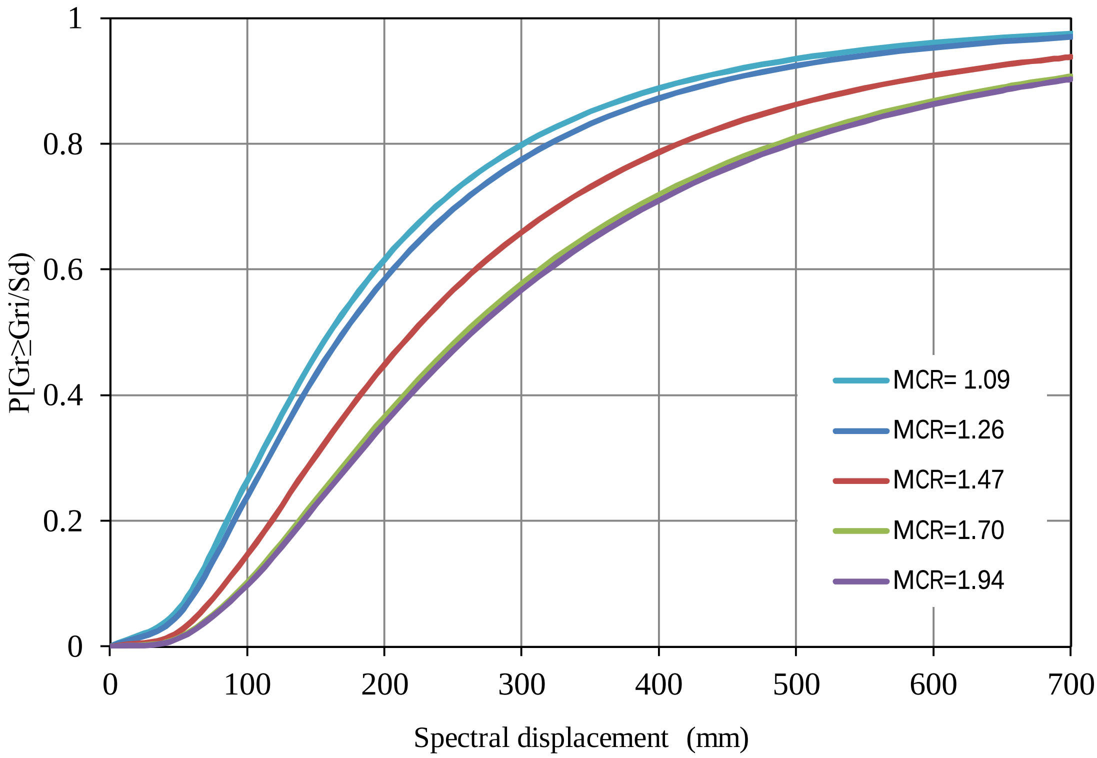


Fig 4.3(a) Fragility curves for 7storey building for Gr3 damage state.

Fig 4.3(b) represents the fragility curves for 7storey building for Gr4 that is complete damage state. These curves also show the similar pattern as above. Gr4 curves are flatter than Gr3 curves because of higher variability associated with this damage state. The total probability is also less than 100 % for the considered range of spectral displacement. It is observed that from MCR 1.09 to MCR 1.26 there is very little change in the fragility .Then up to MCR 1.47 wider variation is observed. The probability of exceedance decreases from approximately 32% to 20 % for a spectral displacement of 300 mm. Then up to MCR 1.7 the variation is lesser and for MCR 1.7 and 1.94 the curves are showing same probability of exceedance. The behaviour of seven storey building is different from that of 5 storey framed building. So there may be some effect of number of storey on MCR of a structure.

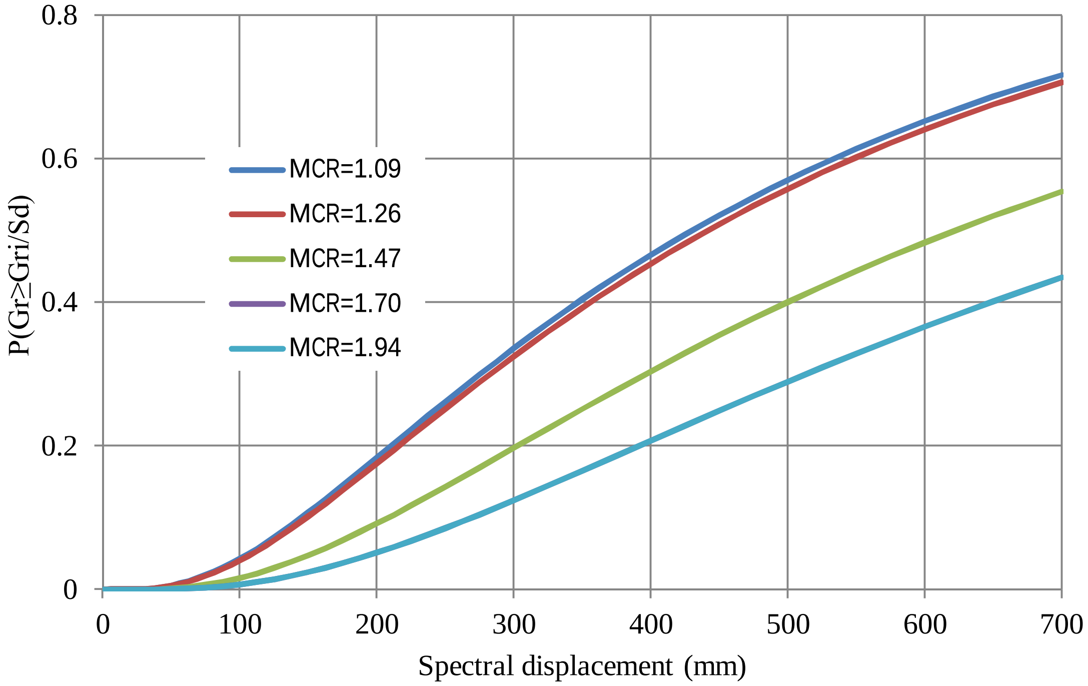


Fig 4.3(b) fragility curves for 7storey building for Gr4 damage state

4.6 PERFORMANCE OF 10-STOREY 3-BAY BUILDING FRAMES

Fig.4.4(a) shows the fragility curves for a 10 storey building with progressively increasing MCR values for Gr3 damage state. It is observed that with increasing MCR values the buildings become safer. MCR 1.47, 1.70 and 1.94 show very little variation in the probability of exceedance. So increase of MCR beyond 1.47 may not contribute much to the safety of the building.

Fig.4.4(b) shows the fragility curves for 10s building with different MCR for Gr4 damage state. It is observed that initially the fragility decreases with increase of MCR up to 1.26 then beyond that the initial portion of the curve shows no difference but as we go on increasing the spectral displacement the probability of exceedance shows a decreasing variation for MCR 1.47. Further increase does not produce any appreciable variation in the fragility of the structure.

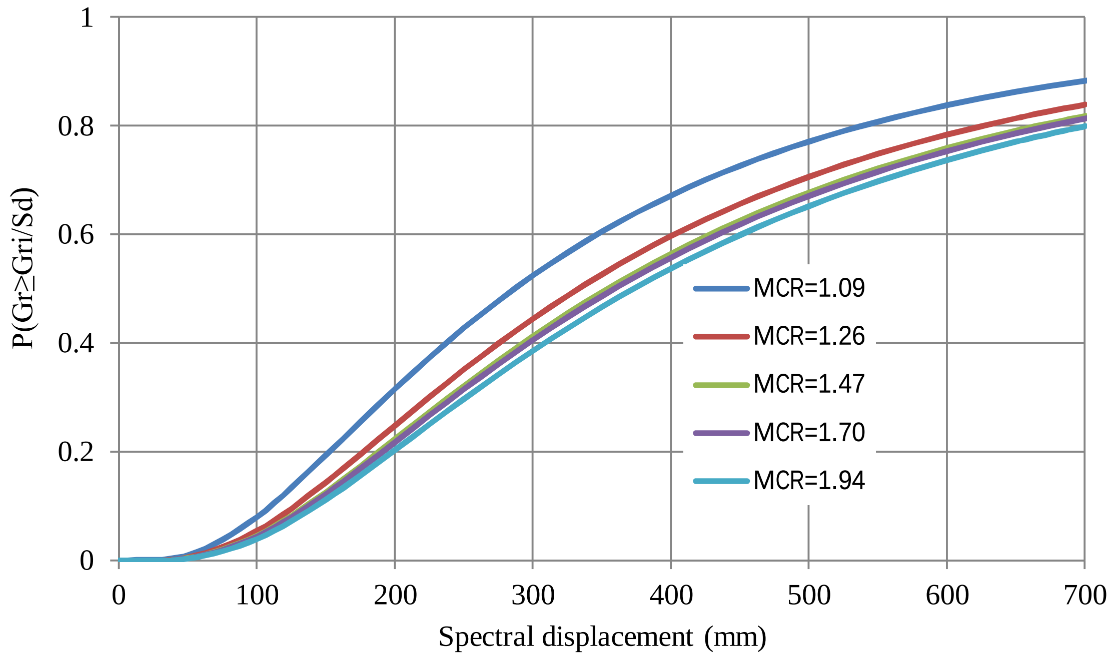


Fig 4.4(a) Fragility curves for 10-storey building for Gr3 damage state

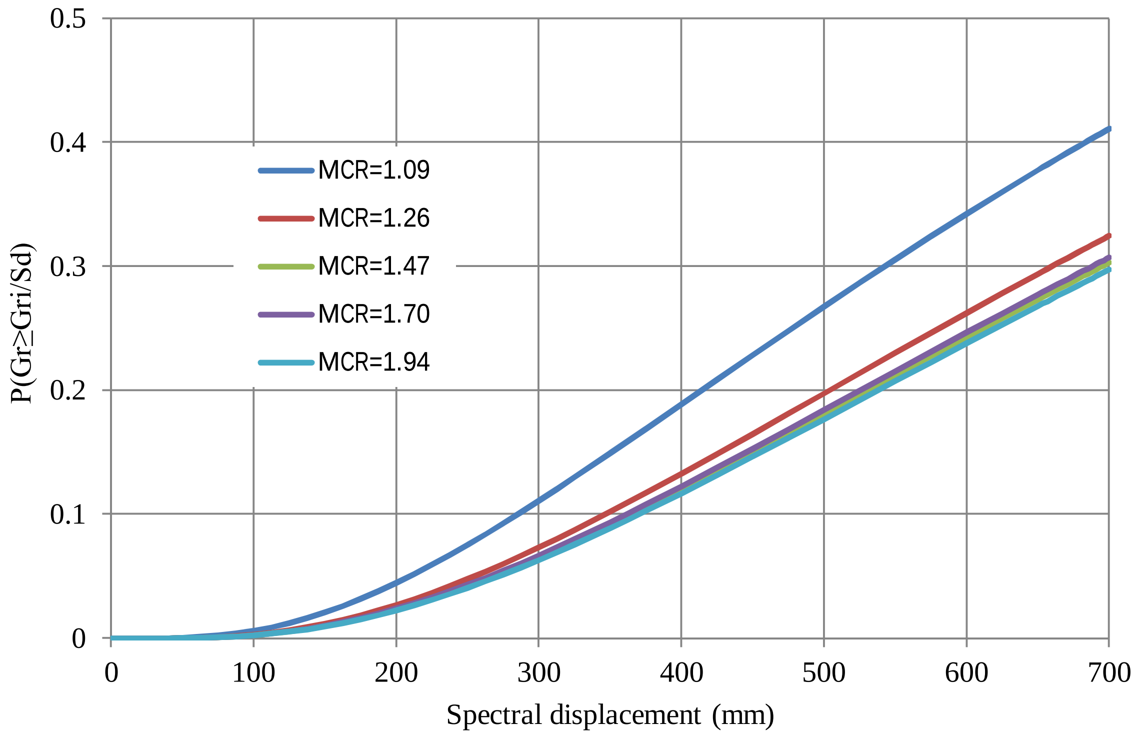


Fig 4.4(b) fragility curves for 10-storey building for Gr4 damage state

4.7 SUMMARY

The performance of regular RC framed buildings is considered by developing fragility curves as per HAZUS (2003). Uncertainties in concrete and steel material properties are considered. 5,7 and 10 storey buildings are modelled for different MCR values by increasing the column reinforcement. Damage state definition is considered from Barbat *et.al.*(2006).Variability parameters are considered as per HAZUS (2003).The fragility curves are developed to find out the vulnerability of buildings without designed as per strong column weak beam concept. The fragility curves for different progressively increasing MCR values are compared for different damage state for a given spectral displacement.

It is observed that a structure designed with lower MCR (*e.g.* 1.09) shows much higher damage probabilities. Fragility of a structure decreases if the columns are made stronger than beams maintained by increasing MCR values.

The results for 5 storey building show little different trend than other two building category (7 and 10storey).The damage probabilities for 5-storey building do not show much variation after MCR 1.26. For 7-storey building frame the probability of exceedance of a given damage state shows a continuous decreasing trend with higher MCR. However the variation after MCR 1.47 is comparatively less. 10-storey building frame also shows the similar trend as a seven storey frame.

Gr4 damage state that considers complete damage of the structure shows flatter slope as compared to Gr3 which is extensive damage state due to higher variability associated with it.

CHAPTER 5

CONCLUSIONS AND FUTURE SCOPE

5.1 SUMMARY

RC framed buildings regular in plan are designed using commercial software STAADpro and modelled using SAP2000. Pushover analysis is done first to study the effect of increase of MCR on ductility and lateral strength of a structure. The effect of increasing moment capacity of column at an expense of extra reinforcement is also observed by obtaining reinforcement ratio as a function of MCR. Reinforcement ratio obtained to achieve different MCR is found to be well within the limits of 6 % as specified in IS 456 and also within 3 % considered for practical purpose.

Even though a structure is designed as per strong-column-weak-beam approach with a suitable MCR, the uncertainties associated with material strength and loads may change the MCR and the structure may collapse in an un preferable failure mode as storey collapse mechanism. So the suitable MCR that is to be proposed should also result in a lower occurrence probability of undesirable failure mode. Therefore to achieve this objective, fragility analysis is being done to study the probability of reaching or exceeding a certain specified level of damage state for a wide range of intensities measures of earthquake. Fragility functions (or curves) are extremely important for estimating the overall risk to the civil engineering structure from potential earthquakes. The economic impact of future earthquakes can also be predicted by this fragility analysis. Furthermore, fragility functions can be used to design retrofitting schemes by carrying out cost/benefit studies for different types of structural intervention schemes. These can also be used to mitigate risk through the calibration of seismic codes for the design of new buildings; the additional cost in providing seismic resistance can be quantitatively compared with the potential losses that are subsequently avoided.

5.2 CONCLUSIONS

The conclusions obtained from this research work is summarised into two parts. The first part deals with the pushover analysis and the second part includes fragility analysis.

Conclusions from Pushover analysis

The non-linear static analysis is done for 5-storey,7-storey and 10-storey building frame with increasing MCR.

i. For the 5-storey building ductility of the structure increases up to MCR 1.47. Increase of MCR beyond 1.47 does not contribute in enhancing ductility.
ii. For the 7-storey building ductility increases continuously with increase of MCR up to 1.94. The rate of increase of ductility is higher indicated by the steeper portion of the curve up to MCR 1.47. After MCR 1.47 the rate of increase in ductility decreases.
iii. The 10-storey building also shows ductility increases up to MCR 1.47 and then it decreases for MCR 1.7. The decrease is may be due to active utilisation of concrete in resisting moment rather than steel. Then for MCR 1.94 ductility again increases. However, highest ductility is achieved at MCR 1.47.
iv. Lateral Strength of the buildings is showing higher values for MCR 1.7 in most cases for both yield strength and maximum strength condition. But since seismic design philosophy demands deformation based design so ductility is most important parameter than strength.
v. So from the pushover analysis it can be concluded that MCR 1.47 seems to be giving good seismic response by enhancing ductility.

Conclusions from fragility analysis

Probabilistic analysis is done to evaluate the damage statistics, and distinguish the buildings on the basis of their relative seismic performance. From the fragility analysis of different building using the capacity curves obtained from pushover analysis the following conclusions can be drawn:

i. The fragility curves indicate much higher damage probabilities for building designed with considering very low MCR value of 1.09.
ii. The incorporation of higher MCR values reduces the damage probabilities irrespective of number of storey and damage level.
iii. For 5-storey building increase of MCR beyond 1.26 does not decrease the probability of damage to an appreciable extent.
iv. For 7-storey building wider variation of damage probability is observed from MCR 1.09 and 1.26 to MCR 1.47 for a given spectral displacement. From MCR 1.47 to 1.70

the probability of exceedance of a given damage state decrease but the difference is comparatively less. For MCR 1.7 to MCR 1.94 almost same damage probability is observed.

v. 10-storey building also shows same trend of fragility curves as of seven storey building frame.

vi. So from the fragility analysis of the three type of buildings it can be concluded that, for up to 5 storey MCR 1.26 seems to be giving a lesser probability of damage. But as number of storey increases, *i.e.* up to 10-storey, MCR 1.47 may be appropriate to have good performance under strong earthquake shaking.

vii. The MCR value depends on number of storeys and it may be higher when number of storeys increases.

viii. A probabilistic framework gives complete insight into the expected performance of a building. So important for performance based seismic design.

5.3 FUTURE SCOPE

- The analysis can be extended with considering more number of buildings with different varying parameters.
- Here only regular RC framed buildings are considered. The analysis can be extended for irregular building having torsion effects.
- Only internal joints are considered in the present work. For external and corner joints also analysis can be done.
- Effect of infill wall can also be evaluated in the analytical models.
- The ground motion parameter can be selected not only as spectral displacement but also in terms of PGA or PGV etc.
- By taking more MCR values the analysis can be done for more number of buildings.

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